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ERRATA.

Proceedings, JANUARY, 1899.

Page 4, second and sixth lines : For "Institute of Engineers," read "Congress of Engineering."

Vol. XXV. FEBRUARY, 1899. No. 2.

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publications.

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American Society of Civil Engineers.

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ON ANALYSIS OF IRON AND STEEL:—Sub-Committee of the American Society of Civ Engineers (of the International Committee on Standards for the Analysis of Iron an Steel, of which Prof. J. W. Langley is Chairman)—Charles B. Dudley, William Metcal Thomas Rodd, A. E. Hunt.

ON UNITS OF MEASUREMENT:—George M. Bond, William M. Black, R. E. McMat Charles B. Dudley, Alexander C. Humphreys.

ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Nobl George S. Webster, O. M. Carter, W. B. W. Howe, Louis C. Sabin, H. W. York.

The House of the Society is open every day, except Sunday, from 9 to 22 o'clock.

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Vol. XXV. FEBRUARY, 1899. No. 2.

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NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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REPORT IN FULL OF THE FORTY-SIXTH ANNUAL MEETING, JANUARY 18th AND 19th, 1899.

Wednesday, January 18th, 1899.—The meeting was called to order at 10.15 o'clock, President Alphonse Fteley in the chair, Charles Warren Hunt, Secretary. Meeting called to order.

The PRESIDENT.—The meeting will please come to order. You know, gentlemen, that under Section 7 of Article VII of the Constitution the polls close at 12 o'clock noon to-day, but if we were to wait until that time to appoint the tellers the counting would take so long that it would bring it too late in the afternoon, and it has been usual on similar occasions to appoint the tellers at the beginning of the meeting, and to ask them to commence their count, with the understanding that anybody who has not voted can do so until 12 o'clock. If there is no objection, we will follow the precedents in that matter, and the chair will appoint as tellers Mr. George S. Rice, of Boston, and Mr. William D. Kelley, of New York. If these two gentlemen want to take hold they will find that our employees will give them all necessary assistance. Tellers Appointed.

Minutes of Last Meeting.

Mr. Secretary, I believe that the first thing in order is the reading of the minutes of the last meeting of the Society.

The Secretary read the minutes of the meeting of January 4th, 1899.

The PRESIDENT.—Are there any remarks or corrections suggested by the reading of the minutes? There being no objection, they will stand as read.

Report of the Board of Direction.

The report of the Board of Direction is now in order, Mr. Secretary.

The Secretary read the Report of the Board of Direction.*

The Treasurer read his report.†

The Secretary read his report.‡

The PRESIDENT.—You have also the report of the Finance Committee, I believe.

The SECRETARY.—Yes, sir. The chairman of the Committee does not seem to be here.

The PRESIDENT.—You might present it now.

The Secretary read the report of the Finance Committee.§

The PRESIDENT.—Gentlemen, you have heard the reading of the reports of the Board of Direction, the Treasurer, the Secretary and the Committee on Finance. What is the pleasure of the meeting in regard to those reports?

A MEMBER.—I move that they be accepted and placed on file.

The motion was seconded and carried.

Report of Committee on Award of Prizes.

The PRESIDENT.—Will you read now, Mr. Secretary, the report of the Committee to Recommend the Award of Prizes?

The Secretary read this report.‖

The PRESIDENT.—This requires no action from the Society, gentlemen, and we come now to the reports of Special Committees. First, we have the Special Committee on the Proper Manipulation of Tests of Cement. I believe the Chairman of the Committee is present. Will Mr. Swain kindly come forward and read the report?

Report of Committee on Tests of Cement.

G. F. SWAIN, M. Am. Soc. C. E.—We submitted this report, Mr. President, because we felt that the Society was entitled to a report from every committee, so that the members of the Society who were interested might know what was being done. A large number of replies to our circular were received, and I have the abstract here, and this, gentlemen (showing bulky package) is the abstract. (Laughter.) To digest that will require some further time on the part of members of the committee. A great many members of the Society, whose information on this subject would be of great value, have not

*See *Proceedings*, Vol. xxv, p. 9.
†See *Proceedings*, Vol. xxv, p. 15.
‡See *Proceedings*, Vol. xxv, p. 16.
§See *Proceedings*, Vol. xxv, p. 20.
‖See *Proceedings*, Vol. xxv, p. 3.

replied to our circular. We shall endeavor, in the next few months, to obtain their replies, and to devote all the time we can to the prosecution of the work.

Prof. Swain read the report.*

The PRESIDENT.—Gentlemen, you have heard the report of the Chairman of the Committee on Cement Tests. The chair is waiting for a motion in regard to it.

JOHN THOMSON, M. Am. Soc. C. E.—I move that the report be received and placed on file.

The PRESIDENT.—Received or accepted?

A MEMBER.—And that the committee be continued.

Mr. THOMSON.—I accept that amendment.

The PRESIDENT.—It is moved and seconded that the report be accepted and placed on file. Those in favor please say aye; contrary, no.

The motion was carried.

Report of Committee on Standard Time.

The PRESIDENT.—We have not got the report from the Committee on Standard Time, but we have a letter from the Chairman, which will be read now by the Secretary.

The Secretary read the following letter:

"OTTAWA, January 16th, 1899.

"CHARLES WARREN HUNT, Esq.,

"*Secretary American Society of Civil Engineers.*

"SIR.—The Special Committee on Uniform Standard Time has been long continued by the Society, and although it has not entirely completed the task assigned to it, sufficient success has attended its efforts to justify the request that the members may be relieved from their duties and the Committee discharged at no distant date.

"The Committee was appointed on June 15th, 1881, on the occasion of the first Convention of the Society held in Canada, and it has been continued ever since; that is to say, for a period of nearly eighteen years.

"The difficulties which necessitated investigation and the appointment of a committee to examine and recommend what course should be followed will be remembered by all who were active members of the Society nearly a generation ago. To-day, owing in a large measure to the intervention of this Society, the conditions are happily changed and it may be well to recall briefly the circumstance which led this Society to take a leading part in overcoming evils which more or less affected the whole community.

"The great net-work of lines of transit with their ramifications, from ocean to ocean which had been and were being established in America created social and commercial conditions which never previously existed in the history of the human race. These new conditions, due mainly to the application of steam and electricity to locomotion and telegraphy, had affected the relations of time and space in a manner which forced itself on the attention, especially of those who were connected with railways.

"The system of reckoning time, inherited from the preceding centuries, proved at this stage in the progress of civilization to be

* See *Proceedings*, Vol. xxiv, p. 192.

Report of Committee on Standard Time (continued).

defective. It baffled business men in their arrangements. It led to confusion and sometimes loss of life. It was consequently productive of difficulties of one kind and another. It, in fact, became of public importance that a body such as the Society of Civil Engineers should direct its attention to the subject and make an effort to discover the means of removing an acknowledged impediment to trade, commerce and general intercourse, and by such discovery to promote our public interests and public safety.

"Many of us are disposed to retain traditional customs on account of their antiquity. We cling with affection even to faults and imperfections made venerable by use and wont. The patient public may be willing to endure evils without enquiring into the cause, but railway men were brought face to face with difficulties in an acute form, which their fathers had never conceived, and they had been compelled to adopt means of meeting them as best they were able. But the remedy applied by individual railway managers only lessened the evil. To operate railways with some approach to safety, each manager arbitrarily set up a time-zero for the running of trains on the lines under his charge. An examination of the guide books showed that there were eighty-nine irregularly chosen railway time-zeros within the United States and Canada, and if to these be added the additional time-zeros used in various cities and towns throughout the country, the need for unification and reformation will at once be manifest.

"These few sentences will give some indication of the problem placed before the Committee for solution. There was an entire absence of uniformity and agreement in the multiplicity of time-zeros, and the work of the Committee was to patiently investigate the whole subject in every detail and ascertain how far it would be practicable to introduce some more simple and satisfactory system of time reckoning; some system which would meet every condition which had arisen or which hereafter might arise.

"The Committee recognized the magnitude of the problem which they were called upon to solve. The enquiry they entered upon demonstrated that it was not a local question, a question confined within the limits of America, but one, indeed, which concerned all nations, especially all nations advanced in civilization; they formed the opinion that in looking for a true solution, which would be permanently useful to the people of the continent, they must find one which would prove effective and acceptable to the whole world. While the existing conditions of society necessitate the preservation of boundary lines between one nation and another, the people of every nationalty have a common patrimony in many things, such as the light and heat of the sun, the seasons, the definite spaces of time which earth and sun, by their marvelous motions, measure off for us day by day. It was desirable and expedient, therefore, that the Committee should take no narrow view of the subject which the Society had confided to them. Patient enquiry and investigation led the Committee to see that the proposition placed before them, was to find the best system of time reckoning for the world; and this Society, if it acted at all, was called upon to take a leading part in a movement to determine a system which, by its simplicity and appropriateness, would commend itself eventually to every nation on the face of the globe.

"This, in brief, is the history of the origin of Standard Time as it is known in America. The Committee, of which I have so long had the honor of being Chairman, was led to adopt an ideal standard. The Society on its part accepted the views of its Committee, and, in the

public interests, has steadily followed a wise course in promoting the great objects aimed at. I do not now propose to recapitulate the action taken year by year. I will only state that the co-operation of other societies was sought and means were taken to invoke the authority of the President and Congress of the United States in order to assemble delegates and representatives of all civilized nations, and through them to advance the movement in this and other countries. Many obstacles have been met with, some of which have been removed, while others still remain to be overcome; but on entering upon so sweeping a reformation, it was not to be expected that all would be plain sailing, or that the undertaking would be entirely accomplished without some difficulty and much delay.

"The accepted scheme may be described as the complete unification of time throughout the globe. Its central feature is the establishment of a universal standard day with its hours numbered in a single series from 1 to 24, as defined by the Washington International Conference. This is recognized as the true common standard of time for all people, for all purposes. It is designed that to this standard all time-reckonings on all parts of the earth's surface, whether on sea or land, shall be directly related. The World's Standard is to supersede all such divisions as at present are designated solar time, civil time, local time, astronomical time or nautical time. The design is that these and all such conventional distinctions are eventually to be abolished and only Standard Time known.

"Much yet remains to be effected, but I can with some satisfaction add that much has been accomplished. We can at least claim that the principles of the reform have been firmly established and that some advance has been made in applying them to every-day life in each one of the five continents. The hour-zone feature of standard time has long been in force throughout North America; it has been accepted in Europe, in India, Japan, Australia and South Africa. True, we have not succeeded in getting the railway managers of the United States unitedly to adopt the secondary feature of Standard Time known as the 24-hour notation, but through the efforts of this Society it has been ascertained that the large majority of managers are individually in favor of the reform. Standard time is obviously incomplete without the 24-hour notation. We find it adopted in its complete form on the Canadian Government Railways between Halifax and Montreal and on every line of railway north of the 49th parallel west of Lake Superior. We find it in Belgium and Italy enforced by statute; we find it in use in India and China. The construction of the great Siberian Railway must have the effect of introducing many reforms in Russia and we cannot doubt that the adoption of standard time in its most perfect form, will, owing to geographical conditions, be one of them. Much the same may be said respecting the unification of time at sea and the change in astronomical time. Seamen are almost unanimously in favor of the one, and the large majority of astronomers throughout the world are in favor of the other, but, so far, united action in either case has not been reached.

"Sandford Fleming,
"*Chairman.*"

The President.—You will notice, gentlemen, that at the beginning of this letter, Mr. Sandford Fleming asks that his committee be dismissed. What is the pleasure of the Society in regard to it ?

Discussion on Report of Committee on Standard Time.

EDWARD P. NORTH, M. Am. Soc. C. E.—Does he ask that it be dismissed now, or does he ask that it be dismissed at some future time?

The PRESIDENT.—(Reading from letter):

"The Special Committee on Uniform Standard Time has been long continued by the Society, and although it has not entirely completed the task assigned to it, sufficient success has attended its efforts to justify the request that the members may be relieved from their duties, and the Committee discharged at no distant date."

It is for this body to judge of what might be called "no distant date."

DESMOND FITZGERALD, M. Am. Soc. C. E.—How many years has that Committee been in existence?

The SECRETARY.—Eighteen years.

Mr. FITZGERALD.—It seems to me, Mr. President, that we should feel lonesome without that Committee.

GEORGE S. MORISON, M. Am. Soc. C. E.—Mr. President, I move that the report of the Committee be accepted and placed on file and the Committee continued. It seems to me that the phraseology of Mr. Fleming does not mean that it be discharged at present, but that he hopes within a year or two to have completed its work. (Seconded.)

The PRESIDENT.—Does anybody desire to speak on that motion?

Mr. JOHN THOMSON.—I would like to ask, Mr. President, if the Committee is yet a complete committee?

The PRESIDENT.—It is not, sir; but that does not impair the motion of Mr. Morison, because there is a cure for that in the powers of the Society.

The SECRETARY.—Under action of the Society some years ago, it is not possible, legally, unless the action is reversed, to accept this as a report of the Special Committee. The rule is: That reports of special committees must be placed in the hands of the Secretary at least thirty days before one of the general meetings, and issued to all members of the Society for the purposes of discussion before they can be accepted, and I do not think that Mr. Fleming intended to do more than to present an informal statement of what the Committee had accomplished—not a report of the Committee.

The PRESIDENT.—I do not see that that changes the position at all; because, assuming even that the Committee has not made any report, it has simply been derelict in its duty, and it does not interfere with its existence in any way whatsoever. So that I do not see that any of the remarks made have anything to do with the motion which has been presented. I will now ask for a vote. The motion of Mr. Morison is that the report of the Committee be accepted and that it be continued.

Mr. MORISON.—I think the proper form of a motion is that the report be received and placed on file and the Committee continued.

The PRESIDENT.—Can you call that a report? That is the question.

Mr. Morison.—It is not a final report. It is a report of progress.

The President.—You have heard the motion, gentlemen. All in favor please say aye; contrary, no.

The motion was carried.

The President.—I believe that one of the members, Mr. E. L. Corthell, desires to read to the Society a communication in regard to a subject that he has very much at heart and considers of some importance. If Mr. Corthell will be kind enough to rise and read his communication, we would be glad to hear it.

Proposed International Congress of Engineering.

E. L. Corthell, M. Am. Soc. C. E.—Mr. President and gentlemen, I am rather unprepared for this. As the President said, it is a subject I have had at heart. That is, it is something that I have been wishing to carry out for several years, on lines on which this Society had requested three years ago when in convention at the Hotel Pemberton. At that time I presented a communication showing some correspondence with engineers in various parts of the world in reference to the establishment of an International Institute of Engineers, but the consensus of opinion was that the time had not then arrived, but that if such a thing was practicable, which was very much doubted generally, it could be led up to by a series of congresses. I am trying to find now the action, if there was any at the time, of this Society, by its Board of Direction, on that subject.* I have not the papers with me; but it is my recollection that at that time the whole matter was referred to the Board of Direction, which decided that there might be a series of congresses leading in that direction. Whether I am right or not I leave you to judge from the records. But during a recent visit of about twenty months in Europe I met a great many engineers, both the Ponts et Chaussées engineers in France, Holland and Belgium, and the civil engineers. I have talked with, have written to, and have letters from, two or three very prominent engineers in the French Government, whose names I do not care to mention, but they

* The matter to which Mr. Corthell refers is as follows:

Action by Board of Direction, October 1st, 1895.

"At the Annual Convention a communication was presented to the Society by E. L. Corthell, M. Am. Soc. C. E., entitled 'Résumé of Correspondence from Engineering Societies Relating to Establishing Closer International Relations,' and on his motion 'the subject covered by this communication was referred to the Board of Direction, with the request that they give the matter careful consideration and action.'

"The Board of Direction has carefully considered the whole matter, and is heartily in accord with what appears to be the prevailing sentiment in favor of cultivating continued closed and fuller relations between the engineering societies of the civilized world. It believes, however, that the true means of attaining the desired end would not be found in an international institute, and agrees with the view so well and clearly expressed on behalf of the German Association of Engineers and Architects by our Corresponding Member, Mr. Charles Otto Gleim, of Hamburg, as to the present desirability of limiting the scope of the movement toward union and following up the course inaugurated at Chicago in 1893 by a series of International Congresses, 'leaving to future development the question whether these congresses will gradually grow into a more permanent organization.'

"In the judgment of the Board of Direction, therefore, it is not expedient for the American Society of Civil Engineers, at this time, to take any steps toward the formation of an International Institute of Engineers and Architects, but that, as a society, it should hold itself in readiness to participate cordially in all efforts to put upon an enduring basis the kindly, social and technical relations which marked the Engineering Congress at Chicago, through a series of future similar gatherings."

International Congress of Engineering (continued).

are men whom you know very well. The subject is one that has been difficult to adjust, and it is a subject which, publicly, I do not care to go into extensively; but any engineer who has been in Europe much and has associated with those two classes of engineers, knows that there is a lack of harmony which might interfere with cordial co-operation in any engineering congress held in Europe. We, in this country, forming a congress, as we did, in 1893, were free from that lack of cordiality, and both classes of engineers in Europe united with us, and we had, as all the world knows, a very successful engineering congress. In fact, if I may speak a word about it, it is generally understood that it was one of the best and most useful of the congresses held in Chicago during those six months. I might say that I have taken the matter up with some of the members of the Institution, in London. They recognized this trouble. About a month before leaving Paris, perhaps in October, I had two conferences with M. Garriel, who is the Government Director of all the congresses that are to be held in Paris in 1900, and he was unable to solve the problem of how the proposition even had better be brought before him. He has the direction of all new congresses, and of all old congresses, such as the Railway Congress, the Congress of Navigation and the Society of Tests. All of those established societies are to hold congresses at the time of the Paris Exposition, and all are to be under the management of this director, simply to arrange for places and times for them. He has nothing to do with the organization and methods of conducting a congress like the Railway Congress; but when a new congress is proposed, then he does have something to do with it, in the way of advice, and, perhaps, of direction.

Not being able to get anything definite from M. Garriel as to how this had best be done, I proposed to the French society, of which I am a member, that the whole question be considered, and I set before them, in a letter which I have here, this trouble which I had found in my conferences with engineers in various parts of Europe. I do not think it is necessary for me to read this letter. It would only take up time. I will give a running account of it as best I can, so that you may understand the situation. After presenting this letter to the Society—and it is a letter that is hardly readable, as I wrote in the most simple language I could, to translate into French, which I did in my halting fashion, to speak before the monthly meeting of that society; it would make you smile if I should read it, and that is one reason why I do not care to do so;—after it was presented to the Secretary of the Society, the day before the monthly meeting, M. Loreau, the President, sent me a kind invitation to take a *déjeuner* at his residence with him on the following day, and there the matter was discussed, as well as I could discuss it with him, I knowing very little French and he knowing no English. But I got the idea, and I suppose he got my idea

(Laughter). I hope he did, at any rate. I went back to my hotel, and wrote a letter to him, meeting some of the points that he had raised. One point was that there were congresses that took up many questions of engineering, and perhaps the necessity or advisability of a general congress was not as apparent now as it was in 1893; and if I may be allowed I will read that letter, which is very short.

"82 Avenue Wagram, Paris, November 3d, 1898.

"Dear Sir:

"Following our conversation of this date at your residence, let me add a few words relating to the proposed Engineering Congress of 1900.

"I am aware that in recent years there have been permanently organized several International Congresses of which some of the features partake of engineering, and that at first thought it might appear that the urgent necessity of an Engineering Congress in 1900 might have passed away, but a little consideration will show that the subjects, purely engineering, taken up by the Congresses already organized cover but a small part of engineering. The Railway Congress deals largely in transportation matters, the Congress of Tests refers only to the characteristics of materials.

"I will now mention a few only of the important Engineering subjects which might profitably be discussed at the proposed General Congress of Engineers, and for which purpose a permanent Congress to be held every two years might profitably be organized without in any way interfering with the subjects of any present organized Congress:

"In the domains of—

"(*a*) Bridges, tunnels, foundations of all kinds, municipal engineering, sewers, water supply, street pavements, rapid transit and development of water powers.

"(*b*) *Mechanical Engineering.*—Machines, engines, boilers of all kinds and classes.

"(*c*) *Mining Engineering and Metallurgy.*—Methods of mining gold, silver, copper, iron ore, lead, salt, etc. The processes of manufacture and refining.

"(*d*) *Electrical Engineering.*—Lighting, tramways, power, telegraphing, machinery of all kinds, not purely electrical science but electricity applied to engineering works and purposes.

"(*e*) *Marine Engineering and Naval Construction.*—The field here is vast and important with the present and future development of steam navigation upon ocean, lakes and rivers, and especially the development of naval vessels, armament and ordnance.

"(*f*) *Military Engineering.*—Fortifications to meet the demands of the ordnance of the present day, the whole field of ordnance for armies.

"(*g*) Engineering Education, which, at present and in the future, is to absorb the attention of Engineering Educators and is of great interest to the engineers of the coming century.

"The field is so broad and important that it can only be glanced over here.

"Then in addition and of still greater importance than purely technical work is the personal and the professional intercourse at the Congress, and the help all would receive from the reports, discussions and decisions of such a Congress on subjects of live and absorbing interest to engineers all over the world.

"I am, sir, yours sincerely,

"Monsieur A. Loreau, "E. L. Corthell.

"*Président de la Société des Ingénieurs Civils.*"

International Congress of Engineering (continued).

In my previous letter to the Secretary I suggested that in order to avoid the obstacle of which I have spoken, the same organization which produced the Chicago congress, the same committees—differently constituted as to the *personnel*, of course—should undertake, in the United States, to form a congress to be held in Paris. But I think the French engineers—I am speaking of civil engineers—are somewhat opposed to that. They think that if a congress is to be held in Paris, they are the men to form it, and I think perhaps they are right. Perhaps it would be more successful, and we would have to conduct this business at arm's length.

Now, that is all I have to say about it, except this, that I saw the Secretary of the French Society the day before I left, early in December, and he told me that the Committee to which my communications were referred for consideration had not acted upon them, but would act very soon. That is the reason this matter has been held back here to this last moment. A week ago I cabled the Secretary asking him to cable me the action of the Society, having in view this meeting. I have not heard from him. Consequently, I think all that can be done here is to decide whether it would be advisable to co-operate with the French Society, or the French engineers, in organizing an International Congress of Engineering in 1900.

The PRESIDENT.—What is the pleasure of the Society in regard to this communication?

Mr. CORTHELL.—I make this motion—and let me say that this is a subject that cannot be very well discussed at this meeting; we have not time for it, and I apologize for bringing the matter before you in this inchoate condition. Therefore, I make this motion, Mr. President: That this communication and the subject of an International Engineering Congress in Paris in 1900, be referred to the Board of Direction.

T. C CLARKE, M. Am. Soc. C. E.—I second the motion.

Mr. NORTH.—Wouldn't it be well to have Mr. Corthell, the promoter of this idea, connected with that consideration?

The PRESIDENT.—I do not doubt that the Committee having this matter in charge will be happy indeed to have the assistance of Mr. Corthell and call him to their council.

Mr. NORTH.—It seems to me that he ought to be chairman of any Committee that is appointed.

Mr. CORTHELL.—Excuse me. I would not like to have that. I am perfectly willing to leave the whole matter, as I did before, to the Board of Direction. If they wish to get from me any ideas about it or to confer with me, I would be very glad to be called before them.

The PRESIDENT.—Is it Mr. North's idea that instead of referring it to the Board of Direction a committee should be appointed?

Mr. North.—I do not wish to antagonize the motion. I withdraw my remark.

The President.—Then the motion of Mr. Corthell stands, to the effect that this matter be referred to the Board of Direction. Those in favor, please say aye; contrary, no.

The motion was carried.

The President.—It is so referred.

H. G. Prout, M. Am. Soc. C. E.—Do I understand that new business is in order now?

The President.—We have one more communication, Mr. Prout, that we should like to present to the meeting.

The Secretary.—I have received the following letter:

Paris Exposition of 1900.

"United States Commission to the Paris Exposition of 1900.
"Chicago Offices, January 5th, 1899.

"Charles Warren Hunt, Esq.,
"*Secretary, American Society of Civil Engineers,*
"No. 220 West Fifty-seventh Street, New York:

"Dear Sir,—As Director of Civil Engineering and Transportation for the American section to the Paris Exposition of 1900, I desire, at the earliest practicable date, to submit the subject in person to your Society, of co-operating with us in the matter of an Engineering Exhibit.

"Will you kindly advise me when I can have such opportunity and I will endeavor to make my arrangements accordingly?

"Sincerely yours,
"Willard A. Smith,
"*Director Civil Engineers and Transportation.*"

The Secretary has communicated with Mr. Smith, but has not since heard from him. Mr. Smith may even be here now, but the Secretary does not know him by sight.

The President.—It is to be to a certain extent regretted that Mr. Smith was not able to come here to give an *exposé* of his views. It seems to me it is a matter of importance, and the Society may like to take some action in regard to it, even in its incomplete state. Will you give me the communication, Mr. Secretary?

It means that the Civil Engineering and Transportation Committee for the American Section of the Paris Exposition would like to ask the Society if it will co-operate with them in the matter of an engineering exhibit. Is this plain enough for the Society to make a motion to the effect that it should be referred to somebody in order to hear Mr. Smith?

Foster Crowell, M. Am. Soc. C. E.—It seems to me that it is a proper matter to be referred to the Board. It would naturally go to them. If a motion is necessary, I move that the communication be referred to the Board of Direction. (Seconded.)

The President.—It is moved and seconded that the communication of Mr. Smith, Director of the Civil Engineering and Transportation

Paris Exposition of 1900.

Committee for the American Section to the Paris Exposition of 1900 be referred to the Board of Direction. Those in favor of this motion please say aye; contrary, no.

The motion was carried.

The PRESIDENT.—Now, Mr. Prout, we should be happy to hear from you.

Twenty-four-Hour System.

Mr. PROUT.—Mr. President, I have a resolution, or a couple of them, that I would like to offer:

Whereas: The use of the 24 o'clock system in the notices of the Society has not accomplished, and is not likely to accomplish, the end for which it was adopted, and tends to create confusion and sometimes ridicule even among members of this Society who have been familiar with it for some years;

Be it Resolved: That it is the sense of this meeting that the use of this system should be discontinued, if this can be accomplished without discourtesy to its former advocates, and

Be it further Resolved: That the Board of Direction be requested to order its discontinuance in the official publications of the Society, whenever, in its judgment, the above condition exists.

I have put the resolution in that shape, sir, in order to get over what seemed to be the main argument last year against the abolition of the 24-o'clock system of notation; namely, discourtesy to the committee which has worked so long and faithfully to introduce it. As you see, Mr. Fleming now seems to realize that not much more can be accomplished in the way of introducing the 24-o'clock system. We, as a Society, have certainly done our part. We have continued to use that notation in our official publications, now, ever since 1886, I believe; and I venture to say that there are not 2% of the gentlemen present here who at this moment think in the 24-o'clock system. I presume that 98% of them have to stop and translate those notices. Certainly, in the aggregate, that is a considerable inconvenience. I think that it is true also that it has a somewhat belittling effect before the world. People who think that we here take it seriously, look upon us, so far as that goes, rather as a set of cranks or faddists; that is, that we are pursuing an absolutely hopeless thing in carrying on this 24-o'clock notation in our official publications; and while I should be the last man to discourage the spirit of martyrdom, still I am not willing to be a martyr without producing any effect. Therefore I have introduced this resolution.

JOSEPH M. KNAP, M. Am. Soc. C. E.—In seconding this resolution, Mr. Chairman, I hardly wish to repeat the remarks I made a year ago in introducing it, and, I think, last year it was defeated, not because the majority were not in favor of the motion, but because, as the mover states, they did not wish to show discourtesy to the committee who have worked so long and faithfully. I have always been in favor of that from the start, as long as there was any hope of its being introduced, especially by the railroads of the country. None of them has intro-

duced the system, and no other body that I know of, with the exception, perhaps, of railroads in Canada, has introduced that system, and its adoption there does not affect us here. It seems to me, sir, that we should vote for this resolution and carry it out, especially as it is left to the Board of Direction to examine into it further and use their discretion in doing away with this system.

Mr. Morison.—Is it not a little premature to vote that a certain practice be discontinued, and then refer the matter to the Board of Direction? That is virtually what this resolution does. In the first place, the preamble stultifies the Society. It says that certain things had not been accomplished which we had hoped to accomplish, and that they are not likely to be accomplished. I do not think we want to say that. It further states that what we have been doing tends to create confusion and sometimes ridicule. I do not think, even if that is the fact, which I do not believe, that we want to confess our sins in that public manner. I do not think that this Society should pass a resolution to criminate itself. I am free to say that I believe most thoroughly in the 24-o'clock system. I also feel that, however such a thing may be in abeyance for a time, it will ultimately come up, and that, in the end, we shall gain by having remained firm to our original purpose. It certainly is a matter which we need not hurry about now. There may be many of us who have to stop and think—many of the Society who have to stop and think—what 22 o'clock means. There are a great many who will save a good deal of time when they figure the difference between 10 and 22, but still, that is the old argument of the good people in Philadelphia who claim that their Quaker origin has enabled them to figure interest and the difference of time between the days of the week and different months quicker than anybody else can. But be that as it may, this matter can just as well be acted upon, if it ever is to be acted upon, at the time that Mr. Fleming anticipates, when he will ask that his committee be discharged; and I would therefore move: That this matter be referred to the Board of Direction to be reported on by the Board at the next Annual Meeting.

A. M. Miller, M. Am. Soc. C. E.—I second, very heartily, Mr. Morison's motion. I believe that this 24-hour system is of enormous importance. I believe that, if we could get the railroads to adopt it, half the accidents would not happen that do happen. There is no question then about A. M. and P. M. I have had experience myself in that, in telegraphing. In respect to steamboats, running down the river, I always make them report when they arrive. At first the telegraph operator refused to receive a dispatch because all my clerk reported was "Arrived 13." I knew where my boat was. I didn't care about the telegraph operator. You will find that all these accidents on the railroads are mainly from P. M. and A. M. We have no P. M. and A. M. in the 24-o'clock system. All astronomers use the 24-hour

Discussion on 24-hour System (continued).

system. I may be considered, by Mr. Morison, a fanatic on that point, but I absolutely say that it is practical, and I believe that we ought to insist upon it. Everybody objected to having the change of meridian at first. It was a long time before that was established. The Detroit people had three times, I think—local time, railroad time and somebody else's time. We finally got that adopted, and there is no trouble about this. It can be done, and it is the simplest way to measure time. God gives us that measure. All astronomers measure the same way, and there is no mistake when you measure the right ascension for a star, so many hours, 24 or 23 or whatever it is; but that finishes the whole business; you know where to find it.

Mr. Prout.—Mr. President, I would like to state just one word in answer to one of the points that Mr. Morison made: namely, that the resolutions point to the discontinuance of this system, and then leave it with the Board. They were so worded designedly. The resolution is: "That it is the sense of this meeting that the use of this system should be discontinued, if this can be accomplished without discourtesy to its former advocates." Then the Board of Direction is requested to order its discontinuance "whenever, in its judgment, the above condition exists;" that is, that it can be accomplished without discourtesy to its former advocates. I see no inconsistency in this.

Mr. Morison.—What that amounts to is, voting now by the Society in Annual Meeting to discontinue the use of the 24-o'clock system. That much is positive. It, then, having taken that action, charges the Board of Direction with some method of doing this—with the task of discovering some method of doing this with the utmost politeness. I do not think that is the proper kind of thing to refer to the Board of Direction. I think the Society should either act at first positively and bear the whole burden of its own discourtesy, if there is a discourtesy, or leave the whole thing to the Board of Direction. I do not like the idea of voting down at a meeting, without notice, a practice which has existed in the Society for a great many years, and then ask the Board to find some means by which the Society will do it politely. I think we should either refer the whole matter to the Board of Direction, or nothing. I therefore make the motion which I did and which may be considered, I presume, an amendment to Mr. Prout's motion. (Seconded.) As I have been asked to repeat that motion, I will state that I move that this subject be referred to the Board of Direction, with a request that they make a report at the next Annual Meeting.

The President.—Do you offer that as an amendment to Mr. Prout's motion?

Mr. Morison.—I will make it as an amendment.

The President.—As an amendment?

Mr. Morison.—Yes.

Mr. Foster Crowell.—Is a remark on the original motion now in order?

The President.—One minute, please. Do you mean the Annual Meeting or the Convention?

Mr. Morison.—The Annual Meeting.

The President.—The Annual Meeting in January?

Mr. Morison.—Yes.

Mr. Crowell.—Are remarks on the original motion still in order, Mr. President?

The President.—Certainly.

Mr. Crowell.—Then I would call attention to the fact that this resolution, while it is aimed at the discontinuance of the notation in the literature of the Society, really seems to be directed to the 24-hour notation in general; and to pass that resolution would look as if it was a declaration, on the part of the Society, that the principle of the 24-hour notation was not a correct one and should be abandoned. I do not know whether the phraseology of the resolution is clearly that; but that is the way it strikes me, and, therefore, I think that if this motion were to prevail it would be doing more than even the author of the resolution intended. The preamble says, "Whereas: The use of the 24-o'clock system in the notices of the Society has not accomplished, and is not likely to accomplish, the end for which it was adopted." That is clear enough. But the resolution that the use of the system be discontinued, implies that we are not in favor even of the idea. I agree with the gentlemen who have spoken, and I have previously expressed myself to the effect, that the ultimate importance of this measure, the practical importance of this measure, is worth whatever ridicule or misunderstanding may be created in the minds of a few people outside of the Society who know that we use this notation. The declaration, however, by the Society that we do not approve of it, even in such a limited way, would go abroad and it would have its effect in discouraging further efforts in this eminently practical direction. I cannot exactly see why so much stress is laid on the difficulty of making the calculations in our minds as to the notices of the Society, when we think of the immensely greater difficulty that is going on all the time, on the part of all the people, in satisfying themselves as to whether 4 o'clock means 4 o'clock in the morning or 4 o'clock in the afternoon.

Mr. A. M. Miller.—It is not 4 o'clock in the afternoon, sir; it is 20 o'clock. I mean 16 o'clock. (Laughter.)

Mr. Prout.—I ask no better argument than that.

Mr. Crowell.—I was, of course, referring to the difficulty outside of the Society. I hope Mr. Morison's amendment will prevail, and that the Society will not take any action at this time; especially as we

Discussion on 24-Hour System (continued).

have just adopted the report of the Committee, which, as far as it goes, is in distinct contradiction of this resolution.

Mr. T. C. Clarke.—Is another amendment in order?

The President.—The amendment is in order; yes, sir,

Mr. Clarke.—Another amendment?

The President.—Not another amendment; no, sir.

Theodore Cooper, M. Am. Soc. C. E.—If my recollection serves me correctly, it was originally ordered in our publications that the two systems be put in all our records—8 o'clock P. M. and 20 o'clock in brackets, or *vice versa*. That was educational, and that was maintained for a number of years. Then by some authority, I do not remember what, I think by resolution; I think the extremists were not satisfied with that, it was ordered that the ordinary time be wiped out. Since then we have been using the 24-hour system solely, and it is not educational. The very purpose which was originally intended was the purpose of educating ourselves. We have failed in that, because now it does not mean anything. You do not have before your eye the two times to get acquainted with them. Now it seems there is a half-way measure here; we can step back to that, and put our time in all our publications according to the ordinary system, 8 P. M., and the 20 afterward in brackets if it is wished to. It will cost very little for type and we will get the benefit of the education. That will not stultify our past action and we will not make ourselves ridiculous, as our friend Mr. Prout says. It strikes me it compromises the two and accomplishes the purpose and at the same time, as I before remarked, it does not stultify our past action.

John F. O'Rourke, M. Am. Soc. C. E.—Mr. Chairman, if there is any one thing I admire in societies or individuals it is courage, and if there is any one thing that I regard as the highest form of courage, it is, after you have fallen or been led into a course of action for which there is no warrant, that you should, when the fact becomes apparent to you, correct it. Now I do not think there is any opinion that is held more strongly in this Society than that the 24-o'clock system is ridiculous, and it is ridiculous for this reason, that the Almighty divided the 24 hours into day and night, and if you send out invitations for a 5 o'clock tea, presuming that you were in ladies' society, you would have none of your callers come at 5 o'clock in the morning. In the same way, there is hardly an incident in a man's daily life or in the actions of a society, that requires the fact to be defined that it is not in the middle of the night or that it is not in the first of the morning. The mere invitation to come to a meeting of this Society implies that it is at the usual hour at which such meetings are held. Such meetings are held not at 8 'clock A. M., but at 8 o'clock P. M.

A Member.—The usual hour would not mean anything.

Mr. O'Rourke.—I beg your pardon, sir; I did not exactly hear.

The MEMBER.—The usual hour would not mean anything.

Mr. O'ROURKE.—The usual hours for doing things are so well fixed that I do not think it is necessary we should make them ironclad. If we want to become very precise I should imagine that we would divide the year up into hours, and call a meeting for the thousandth hour, and then we could dispense with days (laughter).

Mr. Chairman, to go back, as Mr. Cooper did, to the beginning of this movement, it took its rise in the confusion of time tables—by the time tables not stating or showing on their face whether the hour of the train was at night or in the daytime. That was a good point to define the difference between, because when you are dealing with time tables, with trains run all day and all night, and when your day is divided into two parts, the hour of which you are speaking is not so clearly defined. In fact, it is not defined, and a great deal of common sense and proper discretion was necessary in those days to define the two. The solution was to print the numbers darker for the afternoon; that is, the hours after the meridian were printed in heavier type than the morning hours, and then the thing was done with. Anyone who looked at a time table did not have to figure out whether it was dark or light type; he could see it. Anyone who looked at time tables on the 24-o'clock system had to figure it out, and, like one of our enthusiasts on the subject, often figured it wrong. Therefore, Mr. Chairman, I am heartly in favor of not only adopting Colonel Prout's motion, but, for myself, I would go farther—I do not propose to offer that action however—and I would be in favor of dispensing with that thing in a minute. I would get rid of it so quickly that I would want even to forget the fact that this Society for over ten years has been going contrary to common sense and its own beliefs. (Laughter.)

HENRY S. HAINES, M. Am. Soc. C. E.—I hope that I may be pardoned for adding a few words more to this discussion, and I do so because I want to present this subject in an aspect in which I have not yet heard it discussed in this meeting. I may preface the few words I have to say by stating that it is a subject with which I am quite familiar, as I was chairman of one of the committees of the railway managers which introduced what we know as the "standard system of time" in the actual use of the railroads of this country, and subsequently, as President of the American Railway Association for nine years, this matter has been before me very often. I will add that there is one part of this preamble with which I am entirely in accord, and that is that the use of the 24-o'clock system is not likely to be accomplished.

A MEMBER.—I don't agree with you.

Mr. HAINES.—I have told you what my experience has been in the matter. I think anyone who is familiar with this whole subject, so

Discussion on 24-Hour System (continued).

far as the railway system of this country is concerned, will agree that I have had a good deal of experience in this matter. I am not expressing my views as to the advisability, theoretically, of the application of the 24-o'clock system to the uses of the business world. I am only telling you that the railway managers of this country do not believe that it is practicable to apply it to the operation of railway trains. The matter has been before the American Railway Association time and again. It has been brought before that Association through the medium of this Society, and I say, that so far as this Society being able to accomplish the application of the 24-o'clock system to the operation of railway trains in this country, I do not believe that they can do it. But that is not the point of which I wish to speak. I think that what I am about to say would be of more effect because of what I have just stated, and that is that I am not at all in sympathy with this motion, and I will say why I am not. This is not a motion which disposes of the subject in a proper way. There is, as I understand it, a special committee to which this Society has given this subject. That Committee has not yet made a final report. We have simply here a letter from the Chairman of that Committee expressing his individual views, in which he certainly has not asked that the committee may be discharged; but intimates, as I understand his letter, that the time is approaching at which he hopes that it can be discharged, for the reason that it will have accomplished its purpose. Now, why preclude that Committee from making a report on the subject which has been committed to it? Why preclude that Committee from making a report by a resolution of this character? My objection to this resolution is one that I have not yet heard stated, and that is that it is not courteous to the committee that you have appointed, to dispose of the subject committed to it in this way. I do not know what gentlemen are on that Committee, but if there is a member of that Committee present I think he ought to get up and say something in favor of the Committee and of the Chairman who has written that letter, and I do hope that the gentleman who has offered this resolution will withdraw it. As I say, it is not a question of whether you will favor the 24-o'clock system or not. It is a question of whether that is a proper or respectful way in which to treat one of your special committees. (Applause.)

Frederic P. Stearns, M. Am. Soc. C. E.—In the discussions of last year and of this year the merits of the 24-hour system are taken up. Now, I wish to vote against Mr. Morison's amendment and in favor of this original motion; but I am heartily in favor of the 24-hour system. It does not seem to me that it will do one bit of good to continue it in the notices and on our calendar. That is the one point in question at the present time—whether we shall continue it there—and I intend to vote as I have said. Still, I am ready to advance in any way the 24-hour system when it can be done to any advantage.

Henry B. Seaman, M. Am. Soc. C. E.—There seems to be some criticism as to the courtesy of the original motion. I think the amendment is open to the same criticism, that it is discourteous to the Board of Direction to restrict their action to the next Annual Meeting. I think we can obviate both objections by amending the whole—to be referred to the Board of Direction with power to act. They are our representative body. I make a motion to that effect.

The President.—The motion is not in order now.

A Member.—I move that the question be laid on the table. (Seconded.)

The President.—Are there any more remarks?

Mr. Prout.—I wish to add just one word on the courtesy matter. The remaining members of the Committee are Mr. Sandford Fleming, Mr. Charles Paine and Mr. Theodore N. Ely. Two of those gentlemen I count among my closest personal friends, and I should be the last man in the world to do or say anything discourteous to either. The purpose of this was to put it in the hands of our Board of Direction, which we can always trust, to do the reasonable and courteous thing; and I imagine that if it goes to them in this shape, the Board of Direction will see that the thing is engineered through without hurting anybody's feelings, and will let the Committee out in the most graceful manner possible.

J. N. Chester, Assoc. M. Am. Soc. C. E.—I wish to second Mr. Seaman's motion. While another motion was made and seconded between, I should think that was out of order, because neither the gentleman who made the motion nor the gentleman who seconded it was recognized by the chair. Therefore I second Mr. Seaman's motion.

Mr. Morison.—I would rise to a point of order. A motion to lay a subject on the table I think takes precedence of all other motions and is not debatable. I think the only matter before the meeting is the motion to lay it on the table.

Mr. Seaman.—The motion to lay on the table was not seconded.

Mr. Haines.—I seconded it.

Mr. Cooper.—I seconded the motion to lay on the table with as loud a voice as I could.

The President.—Any more remarks, gentlemen? (Question called for.) It is moved and seconded that the matter be laid on the table. Those in favor of this motion please say aye; contrary, no. (Division called for.) I believe we will have to ask the members to stand. Those in favor of laying the motion on the table, please rise.

Seventy-nine members rose.

The President.—Those opposed to laying the motion on the table, please rise.

Twenty-one members rose.

The PRESIDENT.—The motion is laid on the table.

Paris Exposition of 1900.

A short time ago, gentlemen, we read to you a communication from Mr. Willard A. Smith, which was referred by you to the Board of Direction: but inasmuch as Mr. Smith has come in since, and may have some interesting remarks to make, I will ask him to develop the subject of his letter to the meeting.

Mr. WILLARD A. SMITH.—Gentlemen of the American Society of Civil Engineers: I will only occupy your time very briefly, and only that you may have the subject which has been presented to you in the letter fully before you at this time. In the Paris Exposition of 1900 there is the important department of Civil Engineering and Means of Transportation. One of the principal buildings on the Champ de Mars is allotted to this subject. There is also other space incidentally given to it. You will observe that in the title of the department, very properly, Civil Engineering comes first. It is the intention of the Direction of the Paris Exposition, the leading members of which are themselves civil engineers of wide reputation, to make this department a much more important one than it has been at any previous exposition. They are giving especial attention to the historical features. There is a department of the whole exposition called the Retrospective Department, the object being to illustrate the progress in all departments of arts and sciences during the century which is just closing. In this department there are large committees of French business men, engineers and others, representing every department of industry, and among these the very important committee on engineering.

The chairman of this committee has recently issued letters to French engineers throughout the whole nation, asking for their hearty co-operation and defining the outline of the work. Of course the advancement in civil engineering during this century really surpasses anything else that can be shown in the whole exposition. This has been undertaken systematically by the French engineering societies and the various business organizations with which the various engineers are connected. The same work has been undertaken in Great Britain, and on a liberal scale in Germany, where all the engineering and technical societies are uniting for an exhibit somewhat similar, but possibly more complete and larger even than that which they made at the World's Columbian Exposition in Chicago, in 1893, which many of you will remember as possibly the largest and best engineering exhibit which has ever been made anywhere. It is our desire that civil engineering in America should be equally well represented, and I know of no better way than to ask at the outset the co-operation of this Society through a proper committee, or even that this Society should take charge of that subject. I have presented the matter to the Western Society of Engineers, who will undoubtedly co-operate. The method of doing so has not been fully determined. We shall have, it

is assured, some very interesting and important engineering models; notably one of the new drainage canal at Chicago, and in connection with that we hope for working models of all the varied machinery used in its construction, than which probably there could be no more interesting modern engineering exhibit. You of the American Society know better what ought to be exhibited, and, consequently, what can be exhibited, in a fair of this kind. Some small particulars may be of interest to you. On the ground floor of the Department building on the Champ de Mars there are 7 640 sq. ft. In the gallery, which will undoubtedly be devoted largely to models and engineering exhibits, and which is very accessible to light and very desirable in every way, there are 5 166 ft., making nearly 13 000 ft. in that space. That is as much as the allotment given to any country outside of France.

The PRESIDENT.—Does that take the wall surface, Mr. Smith?

Mr. SMITH.—No, sir; floor space only. All wall space is at our disposal, with the single limitation that partitions shall be of a certain uniform height. In the Marine Building, which is situated on the Quai D'Orsay, right on the banks of the Seine, midway between the Champ de Mars and the Esplanade des Invalides, we have on the ground floor a space of 1 862 sq. ft., but our Commission has secured the space and provided for the erection of another building immediately adjoining, to be called the Annex to the Marine Department. The purpose of this building was originally to provide proper quarters for the exhibit of the United States Government Weather Bureau. That will require only one-half of the building. The other half will be at our disposal, provided Congress supplies funds for putting up the buildings which we have to erect, and we shall have there 4 519 sq. ft. of ground space, a space which is very available, in its connection with the marine department and United States Weather Bureau, for certain classes of engineering exhibits. I should say that we are somewhat relieved in the matter of space in the main building by the fact that the large railroad exhibit, the outdoor exhibits, locomotives, cars, tracks and everything of that kind, as well as the entire exhibit of bicycles and automobiles, which promises to be very large, indeed, will be at the Bois de Vincennes, a park with which most of you are acquainted, some nine miles distant from the main grounds, connected by good railroad systems and soon to be connected by another line partially underground, which is now approaching completion, and giving very easy connection. The location is a superb one. It is a beautiful place, the only objection being that it is removed from the main exhibition grounds. As against that, however, we have the fact that there will probably be no other exhibit that will attract the public to so large an extent as the exhibit of automobiles, which are to be shown in operation at this place. There is a great craze on this subject of automobiles. The recent exhibition of

Paris Exposition of 1900 (continued).

1 200 vehicles shows that. So that this location is very desirable for any large railroad or engineering exhibits connected with it. This arrangement will give us more space in the main buildings for the small exhibits. I would like to have a very thorough representation of the most important American engineering work of this century, if it is possible to get at it. We have to labor, however, under this difficulty: As a Commission, we have no money for such purposes. We hope to get—I think there is no question but that we shall get—a reasonable appropriation for the expenses of the loan exhibits, which will not, however, provide for making the exhibits themselves. Among other steps which I have already taken, in the few days since I have accepted the position, is to address a letter to the mayors of the principal cities of the United States, notably the principal ports, suggesting that proper models of their cities be prepared; the scale to be determined afterward, when we find how many wish to do it and how many we can provide space for. Models of this kind, properly made, would be a most interesting exhibit. Certainly one of the City of Chicago would, and I know that a model of Greater New York, showing all the facilities of the harbor and wharves, bridges, terminal facilities, transfer facilities, intramural transit, sewerage system, water supply and everything of that nature, would be of great interest. I think the suggestion will receive very hearty support, and that the city governments will undertake the work under the charge of their own engigineering staff. The suggestion has been well received, not only from the standpoint of advertising each city and as a matter of local pride, but from the fact that such models, when returned to this country, would have a very decided value in the conduct of the engineering department of the city government.

The PRESIDENT.—Excuse me, one second, Mr. Smith. It being 12 o'clock, I have to announce that the ballot for officers of the Society is now closed.

Mr. SMITH.—I have not much more to say, with this exception. The Secretary, in his letter to me, suggests that some information is desirable as to whether any expense to the American Society would be involved in connection with an exhibit of this kind. I can only say that we will undertake the transportation and care of such exhibits and their return. We cannot undertake the expense of providing models, drawings or maps, or anything of that kind, nor, in fact, of the cases in which to exhibit them. But it is my impression that a most desirable exhibit of this kind can be secured at the expense of the local governments or at the expense of manufacturing companies who have certain interests in connection with them. A number of cases of that kind have already come to my observation. We have the privilege of securing from the National Museums, the Smithsonian Institution, or any of the Departments at Washington, the Postal

Department or any other, any models or matters of that kind that we desire. Those were utterly refused to us in the exhibition of 1893, because the Government had its own building and its own exhibit. But we have the right, under the law, to enter any of the Departments at the present time, and call for anything that they have there.

I had the pleasure, on Monday afternoon, of an interview with the New York State Commission of fifteen members to the Paris Exhibition. We did not contemplate the existence of any commissions of this kind, and the difficulty is to find something for them to do. The first question asked is, can they have a space for a New York building? It was with an infinite amount of difficulty that a space for a United States building could be obtained. In the next place could they have a space in the United States building for the State of New York? That is not feasible. The whole plan of the Exhibition is one of condensation. It has been stated repeatedly that only exhibits of the highest nature must be shown. That is the rule of the Paris authorities, and it is certainly absolutely necessary on our part. They have about two-thirds of the space all put together that we had at the Columbian Exhibition in Chicago. The demands for space are very great, and necessarily the amount which they can give to each nation is extremely limited. For that reason we must necessarily study condensation; but I believe that the opportunity for a good American engineering exhibit will be greater than we have ever had before, unless it was in Chicago. There the engineering societies and the engineers in general did not think it altogether important that they should make distinctly civil engineering exhibits, because our business is more broad and we necessarily see the works themselves, and it was not so important that they should exhibit models or pictures or maps of them, and the whole exhibition was necessarily an engineering work. Here, we stand in an entirely different position. France and European countries have that advantage. Their engineering works can be examined by visitors from all parts of the world; ours cannot. The only opportunity we can have for a proper representation of America is to make it in an exhibition of this kind. I desire to say that personally I would be very glad to heartily cooperate with any committee which you might appoint, either in the matter of suggestion or any assistance that I could give, and I should heartily appreciate your assistance.

I did not complete the point which I made in regard to the New York State Commission, which is illustrative of what your Committee could do. I suggested to the Chairman of the Commission—they want a reason for securing an appropriation from the State, of course—that in most departments of the Exhibition they could do absolutely nothing; that the United States Government takes charge of the agricultural exhibits, and also of the exhibit of mines and mining and

Paris Exposition of 1900 (continued).

mineralogy, and of the exhibit of the department of education. You will remember that they had models of the Erie Canal and of the water-ways of the State of New York at the Columbian Exhibition in 1893. With a reasonable appropriation from the State, the State of New York, of course, could make a magnificent exhibit, and I feel free to say to you that the New York State Commission will co-operate with any committee that your Society may appoint, and I believe would undertake to carry through the expense of any exhibit connected with anything in the State of New York. I feel the same way regarding the Massachusetts Commission, from things that have been said there. So that it seems to me that the time is propitious, and that, at the end of the century, American engineers should avail themselves of this decidedly open door to make themselves known to the world. The country has become known to the world lately through a war. President McKinley is exceedingly anxious that we should make a creditable exhibition in what the French call the 'Assizes of Peace' at the Paris Exposition. It follows very properly, and very naturally, that the expansion of our commerce will open fields for American engineers in various countries in the world where they had nothing to do heretofore.

Gentlemen, I thank you for your attention. (Applause.)

Mr. CORTHELL.—Mr. President, may I ask a question of Mr. Smith?

The PRESIDENT.—Certainly, sir.

Mr. CORTHELL.—Will those civil engineering exhibits which you ask us to prepare and send be placed with the engineering exhibits of other countries for comparison?

Mr. SMITH.—They will. The space is arranged in the Engineering and Transportation Building in transverse sections running across the building for each country, and the subjects are to be arranged in sections running longitudinally through the building, so that practically the exhibits of the different nationalities on the same subjects will come in line, and still each nation will have its own exhibits in line, and such features as necessarily come into this department of carriages and bicycles, and exhibits of that kind, will not be interspersed with the real engineering exhibits.

Mr. CORTHELL.—I am very glad to hear that, and I want to say one word on that point, because I am now heartily in favor of it. In 1891 I was in Europe many months unofficially, but doing a good deal of missionary work, if I may so term it, and was in consultation occasionally, in different countries, as we met, with the United States Commission that was there for the purpose. I saw a great many engineers from various countries, Germany, France, Belgium, Holland and Great Britain, to endeavor to get them to send to the Chicago Exposition original exhibits, the very sort of things that Mr. Smith is asking us to send. Yes, they would send them. In the meantime the

English Commissioner and the German Commissioner, in my absence and unknown to me, stated that they wanted the exhibits from England and from Germany with their own exhibits. So the engineer told me: "If that is the plan we will send nothing. We do not propose to have our models and maps lost in the great mass of matter which will come over from our own country." I am very glad to know that there will be a different arrangement, so that if we send our exhibits, showing our Chicago Channel, for instance, showing the exhibits of the docks here, it will be placed where anybody can step into another department nearby and see what Germany has done or what Holland or England has done. So that there will be an engineering competition, and I think that we can afford to enter into it, from what I have seen in Europe in the last few years.

Mr. North.—As one of the Committee that got up the exhibit for the Paris Exposition in 1878, I think there are one or two things to which attention should be called. First, it is absolutely necessary, if the exhibit is to be of any use to the United States, that the man in charge of it should be an engineer. We had money enough to get up an exhibit which was without a peer, the best examples of the work of the best engineers in the world: but there was nobody there to explain it, and it did us so little good that between lack of knowledge and lack of interest on the part of our Commission, we got just as much reward as the British Institution of Civil Engineers got for toting there a set of their *Minutes of Proceedings* when the fair was about half through. We were not able to keep a man there to explain our exhibit, and we were without sufficient backing from the American Commission. Now, I take it from Mr. Smith's coming here that we will not stand in danger of not having backing. But the Society, or members of the profession, whether they belong to the Society or not, should not send over exhibits unless there is somebody there who has knowledge of engineering matters and can explain them. Of course, a large manufacturing concern will keep one or two agents there as a trade matter. But, for instance, if maps or models showing the Croton Aqueduct supply were sent over, not by the city, but by the engineer who built them, he would be without resources to keep a man there to explain them, and the exhibit would be virtually thrown away. I think that the Society should have a clear understanding on that point before they invite people, if they conclude to exhibit.

Mr. Morison.—I believe I was Chairman of the Committee of which Mr. North has just said that he was a member. I do not want to say much, but I think the experience of that Committee and of that exhibition of 1878 shows that it is well worth while to exhibit American engineering at the Paris Exhibition of 1900. But I think it also indicates that it is probably not expedient to appoint a committee to be specially charged with getting up such an exhibition. So far as relates to the

Paris Exposition of 1900 (continued).

objection which Mr. North has raised, that we cannot have an engineer there, or that we may not have an engineer there to represent us, that certainly was a very serious thing in 1878. I do not think it will be a serious thing in 1900. I know Mr. Smith. I know what he did in the Chicago exhibition in 1893. Mr. Smith was in charge of the Department of Transportation there, and there was no department in the exposition as well handled, in my judgment, as Mr. Smith's department. (Applause.) I feel that the fact that Mr. Smith is going to be in charge of this department of the American exhibit at Paris is the best assurance we can possibly have that engineering will be properly taken care of, and I think individual members of this Society who feel disposed to take any action in that way, can entrust their interests to Mr. Smith quite as well as they could to any committee which the Society could appoint. (Applause.)

Robert Cartwright, M. Am. Soc. C. E.—There is one feature, I think, that my brother North has neglected, in relation to having an engineer there to explain the exhibit. I am afraid that unless that engineer was a polyglot there would be a great many who would be unable to understand his description. For instance, we might have a Chinaman come along and want to know about something, and my friend North, if he talked the Chinese language, might explain it. Wouldn't it be well to keep that consideration in mind and have brother North prepare himself and study the different languages so as to go over in that capacity. (Laughter.)

Mr. North.—When I go to a foreign country the language of which I do not understand, I generally hire an interpreter.

The President.—Is there any other business that any other member would like to present to the Society?

Legal Protection for Civil Engineers.

Mr. S. C. Thompson.—I have a matter to present to the meeting which you have heard before, modified slightly, but covering in substance the resolution presented a year ago:

In consideration of the fact, That the engineering profession and the people of this country have no legal protection against any incompetent or unscrupulous person who chooses to advertise or sign himself with the title of Civil Engineer, thereby reflecting upon and injuring the entire profession; and

In consideration of the fact, That it has been found advantageous in most States to grant legal protection to members of the legal and medical professions; therefore,

Be it resolved, That it is the sense of this meeting (the Annual Meeting of the American Society of Civil Engineers), that judicious legal restrictions against the unauthorized and improper use of the title of Civil Engineer are desirable.

I would like to say, in submitting this resolution, that I hope the gentlemen will see what it says, and not what they think it intends to do. It does not commit the Society to any legal action or anything of the kind, because I fully appreciate that that is impossible. I have made no effort to spring this upon the Society or to have it come without being expected by anybody. I have talked the matter over with

some, and I have found, especially among the younger men, that they do not feel that they have any inducement, in a general way, to take up the profession and spend years in preparing for it, and then find that anybody, without any preparation whatever, has just as good a chance and just as good a legal right to use that title. I therefore present this resolution, and I move that it be adopted.

The PRESIDENT. – I do not hear that motion seconded.

A MEMBER.—I second the motion.

PALMER C. RICKETTS, M. Am. Soc. C. E.—Mr. Chairman, when this motion, or a motion which was practically its equivalent, was made last year, one of the Past-Presidents of the Society, if I remember correctly, Mr. Morison, rose and said that he was not quite certain that he had any right to call himself a civil engineer. Now I think that we have not yet got to the time in this country when we can settle the question legally. I think it will have to work itself out for some years yet. Of course, from my position perhaps as a director of a civil engineering school, I ought to take the other side of the question; but my common sense will not permit it. I do not think the Society had better mix itself up with the question for a while yet.

Mr. CARTWRIGHT.—Mr. President, who is to confer the degree of civil engineer, unless it comes from some technical institution or something of that kind? Now, I honestly confess that I graduated from no college, except the college of adversity, and I contend that God Almighty makes the engineer. It is not the school. If the man is an engineer he will make his mark himself. Brother Morison says he does not know whether he is entitled to use the degree of civil engineer. He might just as well take up theology, and I don't know but what he would be as good a success. (Laughter.) I think those things work themselves out. A man's work will qualify him to be called a civil engineer.

Now, another point—we had this fight a year ago, if you remember —there is one thing that I would like to put right here to this Annual Meeting. We want relief from that more than anything else, and that is the subject of acoustics. My brother Morison being here gives me courage to speak. Every man who has gotten up and spoken this morning has been invited to go up to the front that we all could hear, with the exception of one or two. Mr. Smith was remarkably distinct. I may be growing deaf. I don't know but what I ought to offer a resolution that the Society furnish a megaphone that may be used by the Secretary or any of the speakers. I am heartily in accord with everything that has been done. I am with Mr. Corthell's motion. If he would only include in all his varied descriptions of engineering—if he would only include acoustics, I am with him. I am with my friend Mr. Smith in regard to the exhibition, if he would only make a model of this room and send it to Paris as a specimen of a civil engineer's auditorium. We can do that and make money out of it. I am per-

Legal Protection for, Civil Engineers (continued).

sonally satisfied that Mr. Wanamaker would gladly furnish us with drapery, marked with mottoes and his own advertisement and do it gratis, so that we can have "Glod Bless our Home," "Engineers, Love one Another," and mottoes like that before us. I know that Wanamaker would furnish the material. For gracious sake let us do something to cure this miserable auditorium, so that persons can be heard. I spoke on this last year; so did Mr. Morison. I hope that a committee will be appointed to do something; either put up draperies or arrange it in some way so that we can be heard. I confess I have heard very little that our President has said. My hearing may have become dull; but it seems to me I am no worse off than a great many of you.

The PRESIDENT.—Any more remarks, gentlemen, on this question?

Mr. MORISON.—Mr. President, I move that the matter be laid on the table. (Seconded.)

The PRESIDENT.—It is moved and seconded that the matter be laid on the table. Those in favor please say aye; contrary, no.

The motion was carried.

Resolution Standard Time Committee.

Mr. O'ROURKE.—Mr. Chairman, I would like to offer a resolution in regard to the time question:

Resolved: That the Committee on Standard Time be requested to make a final report to the Society at the next Convention.

(Seconded.)

The PRESIDENT.—You heard the motion, gentlemen. I will read it

Resolved: That the Committee on Standard Time be requested to make a final report to the Society at the next Convention. Is that resolution seconded?

Mr. JOHN THOMSON.—I second the motion, Mr. President.

The PRESIDENT.—What is the pleasure of the Society in regard to this? (Question called for.) I do not hear any remarks. I will ask for a vote on this question. Those in favor please say aye; contrary, no. The Chair cannot decide the vote from what we have heard, and I will ask the members to stand. Those in favor of the resolution please stand.

Forty-eight members rose.

The PRESIDENT.—Those who are not in favor of the resolution please stand.

Forty-three members rose.

The PRESIDENT.—The resolution is adopted.

Mr. O'ROURKE.—Do I understand that the resolution is carried?

The PRESIDENT.—It is carried; yes, sir.

Any more business?

Proposed Meetings for Juniors.

FRANCIS COLLINGWOOD, M. Am. Soc. C. E.—Mr. President, I desire to make a few remarks, and then to offer a resolution. In establishing the fund for the advancement of the interests of the Society by trying to induce an interest among the Juniors of the Society, I was really in hopes that we would have reached a result more positive than we have thus far. I have watched with a good deal of interest

the progress that has been made in the Institution of Civil Engineers among the Juniors, or the Students, as they are there called, and I cannot see why we should not have——

A Member.—We cannot hear a word.

The President.—Please step up this way, Mr. Collingwood.

Mr. Collingwood.—In reference to the advancement of the Society by getting some young men to join the Society, that was the object I had in mind in establishing the Junior prize, and I confess that I have been disappointed in seeing that the result that was aimed at has not thus far been reached, seeing what has been done in the Institution, where the Students have their regular meetings, and where there are a good many very valuable papers presented. I cannot see why we should not reach the same result here. We have just as smart men, and they can do the work if they will. I believe it is only necessary to get the thing started in order to have a great deal more done by the Students than we have yet attained. It has seemed to me we might do something by copying—I do not believe in copying usually—by copying what they are doing there; that is, in trying to get the Students, or, as we call them, Juniors, to come together by themselves and have their own meetings. I see no reason why we should not do that. They may not be Quaker meetings. They could have meetings once a month, or once in two months—whenever may be thought best. We know that a Junior will speak to a Junior when he would not speak to those who are older. I remember very well the paper I first presented to the Society. I did not want to present it. I thought it was commonplace, and yet it was all work that I thought had not been done before. To my astonishment, the paper was published abroad, and was the basis of considerable comment. I believe there is among the Juniors of the Society a great deal of original interest, if they would only bring it out; and the object of these meetings, if they are held, should be to get the Juniors to come together and talk among themselves, and occasionally evolve a paper that would be of value to the whole Society. I present the matter in that way, and I would like, if the Society thinks well of it, that some action be taken for meetings of the Juniors as such. I make that as my resolution.

Mr. Corthell.—I second the resolution.

The President.—Have you got your resolution in writing, Mr. Collingwood?

Mr. Collingwood wrote out his resolution and handed it to the Secretary.

The Secretary.—The resolution of Mr. Collingwood is as follows:

Resolved, That it be recommended to the Board of Direction that steps be taken for holding meetings of the Juniors of the Society for the reading and discussion of papers.

The President.—You have heard Mr. Collingwood's motion, which has been properly seconded. Does anybody desire to speak on that

motion? We might like very much to hear from some Junior. Those in favor of Mr. Collingwood's motion, please say aye; contrary, no.

The motion was carried.

The PRESIDENT.—Is there any more business before the Society from any of the members? Mr. Secretary, if you have an announcement to make, now is the time.

Invitation from Engineers' Club.

The SECRETARY.—I have the following letter from the President of the Engineers' Club addressed to the Secretary:

"JANUARY 12TH, 1899.

"Mr. CHARLES WARREN HUNT,
"*Secretary, American Society of Civil Engineers,*
"New York:

"DEAR SIR,—I have the honor to invite the members of the American Society of Civil Engineers who may attend the Annual Meeting of the Society, to visit the House of The Engineers' Club, 374 Fifth Avenue, near Thirty-fifth Street.

"Accompanying this are a quantity of signed cards, extending the privileges of the Club for a period of ten days to such of the members of the Society as may have their names filled in by you.

"I would thank you to also announce that ladies, accompanied by members of the Club, or by our guests with these cards, are invited to visit and make use of the Club between the hours of 11 and 3 o'clock.

"We shall be glad to see and welcome as many as may find it convenient to call.

"Yours very truly,
"JOHN THOMPSON,
"*President.*"

Any member of the Society who will apply to the Secretary can secure a ten-day card to the Engineers' Club.

Programme of Annual Meeting.

I suppose it would be well, Mr. President, to announce something about the programme. The first thing is that immediately after the adjournment of this meeting a lunch will be served in the room just below this, and a meeting of the Board of Direction of the Society will be held at half-past fourteen. (Laughter.) The two excursions which have been arranged for this afternoon, one to the New East River Bridge, and one to the docks, piers and bulkhead wall will start immediately after lunch. I presume that they will leave about 14.15. The excursion to the New East River Bridge will, at all events, leave at that time and will take the Fifty-ninth Street cross-town cars. If all members who desire to make this trip will communicate with the Secretary immediately after the adjournment of this meeting, or with Mr. E. G. Freeman of the Committee, it is probable that special cars can be arranged for, so that the party will not have to split up; but we will have to know how many people will go, and whether they will go promptly at the hour specified, in order to secure those cars.

Mr. DESMOND FITZGERALD.—May I ask a question? What time will that excursion return?

The SECRETARY.—What time do you think we will get through, Mr. Freeman?

Mr. E. G. Freeman.—About half-past four.

A Member.—Half-past sixteen.

Mr. FitzGerald.—Will it be possible for any member who wishes to see that interesting work to be able to go there after half-past—what is it—sixteen?

Mr. Freeman.—They can come at any time. It is only that it will be more convenient for those going this afternoon if they could go together. But at any time if they care to go down I am sure the engineers will be glad to extend all the courtesy possible.

Mr. FitzGerald.—How long will it take to go down there?

Mr. Freeman.—It will take three-quarters of an hour from here.

Mr. FitzGerald.—And you will be there probably until after four o'clock?

Mr. Freeman.—Yes.

The Secretary.—Mr. Chandler Davis, Assoc. M. Am. Soc. C. E., will have charge of the other excursion to the docks, and should be notified by members who intend to go.

At 21 o'clock this evening there will be a social meeting at this house. The programme is all printed and has not been changed. Tickets for this reception can be obtained from one of the Secretary's assistants, Mr. Burke, by those who have not secured them. Tickets will also be required on the steamer excursion to-morrow, and can be secured at the same place. The only change in the programme for the excursion to-morrow, so far as I know, is the substitution of another steamer for the *Laura M. Starin*. The *Valley Girl* is the boat we will take.

At 20.30 o'clock to-morrow, Thursday, at the house, Mr. E. Wegmann, M. Am. Soc. C. E., will deliver a lecture on "The Old Roman Aqueducts," and after this lecture, which will be illustrated by stereopticon views, there will be a "smoker."

For the excursion on Friday to the new Croton dam, no tickets will be required. There will be two special cars attached to the 10.45 train on the New York Central and Hudson River Railroad from the Grand Central Station, and if the members will present themselves in time to catch that train they will not have to procure tickets from anybody. They should notify the Secretary's assistant of their intention to go, because Messrs. Coleman, Breuchaud & Coleman have to prepare to take care of the proper number, both on the train and at the works, and it is an all-day excursion; so anybody who intends to go should notify us as soon as possible. I think that is all I have to say on the programme.

I am requested by a member of the Institution of Civil Engineers to say that it is desirable that a conference of the Members of the Institution who happen to be present at this meeting be had, and that that meeting will be called directly after the adjournment of this meeting in the office of the Secretary. He desires me further to state that it will only take a few moments to transact what business would be brought before them. Mr. Corthell has asked me to call the meeting.

The President asks me again to announce that the meeting of the Board of Direction of the Society will be held at 14.30 o'clock in the Secretary's office.

I have to announce the following deaths:

Announcement of Deaths of Members.

Samuel Killebrew, elected Member, September 2d, 1885; died January 9th, 1899.

James Duane, elected Member, March 2d, 1892; died January 12th, 1899.

I might also announce that the next Annual Convention of the Society will be held June 27th to 30th, 1899, at the Stockton Hotel, Cape May, N. J.

Report of Tellers.

Mr. President, that is all I have, except the report of the tellers.

The President.—You will please read the report of the tellers.

The Secretary read the report of the tellers.*

The President.—You have heard the report. I now declare that the members whose names you have heard are elected to the various offices for which they have been respectively named.

President FitzGerald takes the chair.

Before leaving this chair, gentlemen, I only want to say two words —to thank you for the uniform courtesy and kindness you have shown me during the last year, and it now becomes my pleasant duty to introduce to you the new President, Mr. Desmond FitzGerald, of Boston. (Applause.)

Mr. Desmond FitzGerald.—Gentlemen, I do not know what is expected of me on this occasion. I certainly did not know that any remarks were in order. But I do want to thank you for the great honor conferred upon me by your votes to-day. When I was told that my name was to be used by the Nominating Committee I must say that I began to look around to find some basis for so great an honor, and the more I seemed to grope around, the less I seemed to find. A strange thing happened, however, in the course of a few days. Another wire informed me that my youngest daughter had added the third generation to our family. (Applause.) It was a twelve-pounder, and then I began to feel a little consolation, having at last found some foundation for aspiring to an office of this dazzling height. But seriously, gentlemen, I believe that this is one of the greatest honors in the gift of the American people, because I believe, first, that this Society is necessarily the greatest society of engineers in this country, and then that it represents what, if not already the greatest profession in the world, is fast gaining the top of the ladder. There are many indications of this movement in these times. I must confess for one that I never look upon the work of my brother engineers, particularly in other specialties than my own, without wondering at the patience, intelligence and audacity that are brought to bear upon the solution of the great engineering problems of the day. You will find these quali-

*See *Proceedings*, Vol. xxv, p. 5.

ties shown in every branch of the profession. We cannot examine the magnificent work of our bridge engineers without wondering at the quiet but remarkable progress they are making in their recent undertakings. The same may be said in regard to all forms of sanitary science, sewerage, water supply, and other municipal work. Then we have advances in river and harbor works and magnificent strides in the designs of mechanical and electrical engineers in all the different ramifications of those important branches of the profession. Aside from the work of the engineer in active practice, we must admire the silent but effective work of the professors and other teachers in our colleges and technical schools, who are doing so much to improve the profession in the preparation of those who are to succeed us.

But, gentlemen, I do not think that any extended remarks are expected on this occasion, One thing has pleased me very much as I have been sitting here to-day, and that is, to find that so much thought has already been given to the Exposition of 1900. I have been thinking a good deal about this coming event, and I believe that I express the hope of many members of the Society when I say that I trust we shall have a steamer of our own in 1900 to take over a large delegation of engineers to this exhibition, and I do not doubt that it will be a larger representation than that which visited the last exposition. It is not too early to begin now the work of preparation, and with your help I intend to take the necessary steps to bring this about. The Society owes a debt of gratitude to Mr. Corthell for his interesting and valuable work which he has pursued so steadily against every adverse wind. (Applause.) I believe that the seed which he has planted will bring forth abundant fruit. It is gratifying to find that the direction of the exhibits proposed for the exposition are in such good hands.

One word more, gentlemen. This is comparatively a young society; at least, when you look across the borders of Massachusetts and consider the age of the Boston Society of Civil Engineers. Recently that society held in Boston its fiftieth anniversary, and a most enjoyable occasion it was. One of the interesting features was the presence on the platform of one of the original founders of the society. The whole tendency of the meeting was toward a long look into the past, and we can hardly do this without realizing that our proud position to-day as a profession is largely due to the splendid work of those who for so many years, against every discouragement, prepared the work for us. I trust that when this Society celebrates its semi-centennial, it will be as interesting an occasion as the recent one in Boston, but I know you are looking toward the lunch room and the excursions to follow, and I will not detain you longer. (Applause.)

As there is no more business, gentlemen, a motion to adjourn is in order.

On motion, the meeting adjourned.

EXCURSIONS AND ENTERTAINMENTS AT THE FORTY-SIXTH ANNUAL MEETING.

Wednesday, January 18th, 1899.—After the Business Meeting, lunch was served at 13.30 o'clock in the Lounging Room, after which most of those present joined one of the two excursions to points of interest. Large parties were taken to the following places: To the New East River Bridge, under the guidance of E. G. Freeman, M. Am. Soc. C. E., and Edwin Duryea, Jr., M. Am. Soc. C. E.; and to the docks, piers and bulkhead wall of the Department of Docks and Ferries, under the guidance of Chandler Davis, Assoc. M. Am. Soc. C. E.

At 21 o'clock an Assembly was held in the Society House, which was largely attended.

Thursday, January 19th, 1899.—The day was devoted to an excursion on the steamer *Valley Girl*. The boat left the pier at the foot of West Fifty-seventh Street, at 10.30 o'clock, and after calling at the pier of the Albany Day Line, foot of West Twenty-second Street, proceeded down the river and through New York Bay and the Kill von Kull to Elizabethport, N. J., where, by invitation of Lewis Nixon, President, the works of the Crescent Ship-Yard were inspected.

Lunch was served on board, on the return trip, and a stop was made at the Brooklyn Navy Yard, where by kind permission of Commodore J. W. Philip, U. S. N., Commandant, the *Indiana*, *Massachusetts* and other ships were visited. Facilities for the inspection of all points of interest in the Navy Yard were afforded, through the courtesy of Naval Constructor Francis T. Bowles, U. S. N., and Civil Engineer P. C. Asserson, U. S. N., M. Am. Soc. C. E.

A final landing was made at the Recreation Pier, foot of East Twenty-fourth Street, Manhattan, where the party disbanded. The party numbered about 300 members and guests.

At 20.30 o'clock, E. Wegmann, M. Am. Soc. C. E., delivered a lecture on "The Old Roman Aqueducts." The lecture was illustrated with stereopticon views.

After the lecture there was an informal "Smoker" which was enjoyed by about 170 members and guests.

Friday, January 20th, 1899.—By invitation of Messrs. Coleman, Breuchaud and Coleman, a party of about 150 members and guests visited the New Croton Dam. The party left the Grand Central Station in special cars attached to the regular train of the New York Central and Hudson River Railroad, leaving at 10.45 o'clock. At Croton Landing conveyances were provided for the drive of 2½ miles to the Dam. Lunch was served in the office of the contractors.

Every facility was afforded for the inspection of the work in progress. The party returned to New York by the same route and arrived at 18 o'clock.

The names of 316 members, of various grades, in attendance at the Annual Meeting, excursions and Assembly, are given below. The list is, however, incomplete on account of the failure of a number of members to register, and does not include any of the guests of the Society or of individual members.

Allen, Calvin H....New York City
Amweg, F. J... Philadelphia, Pa.
Andrews, Horace ..Albany, N. Y.
Auchincloss, W. S..Summit, N. J.
Aycrigg, William A. New York City
Aylett, Philip.....New York City

Bacon, John W..Danbury, Conn.
Bailey, George I....Albany, N. Y.
Baldwin, Fred H Bayonne, N. J.
Barnes, William T..Boston, Mass.
Barnsley, George T Bethayres, Pa.
Bates, Charles, J. Highwood, N. J.
Bauer, J. L.....New York City
Belden, Edgar T....Nazareth, Pa.
Belknap, W. E....New York City
Belzner, Theodore. New York City
Bensel, J. A.....New York City
Berger, Bernt ....New York City
Bettes, C. R. Far Rockaway, N. Y.
Betts, F. K .....Valhalla, N. Y.
Bigelow, W. D ...New York City
Bishop, George H.
Middletown, Conn.
Bissell, H...West Medford, Mass.
Blakeslee, Clarence
New Haven, Conn.
Boecklin, Werner, Jr.
New York City
Bogue, V. G.....New York City
Boller, A. P .....New York City
Brackenridge, J. C. Brooklyn, N.Y.
Bradley, C. W.....New York City
Bradley, W. H.. ...Boston, Mass.
Breithaupt, W. H..New York City
Briggs, Josiah A. New York City
Brinckerhoff, A. G. New York City
Brinckerhoff, H. W..New York City
Brown, Thomas E.
New York City
Brown, LeGrand. Rochester, N.Y.
Brown, W. P.....New York City
Breuchaud, J. ....Yonkers, N. Y.
Buck, L. L .....New York City
Buck, R. S .....Niagara, N. Y.
Burdett, F. A....Brooklyn, N. Y.
Bush, H. D....Springfield, Mass.

Carll, D. S....Washington, D. C.
Carney, Edward J. New York City
Carr, Albert....East Orange, N. J.
Cartwright, R....Rochester, N. Y.
Cattell, W. A. Richmond Hill, N. Y.
Chase, R. D.....Brooklyn, N. Y.
Chester, J. N.....Brooklyn, N. Y.
Christian, G. L....New York City
Christy, George L. New York City
Clapp, L. Russell
Hempstead, N. Y.
Clapp, O. F.....Providence, R. I.
Clark, L. V. Jr..Philadelphia, Pa.
Clarke, T. C.....New York City
Coffin, Amory.....New York City
Coffin, Freeman C..Boston, Mass.
Cogswell, W. B...Syracuse, N. Y.
Colby, S. K.....New York City
Cole, Howard.....New York City
Collingwood, F...Elizabeth, N. J.
Cooper, Theodore..New York City
Cornell, G. B.....New York City
Corthell, E. L.....New York City
Coverdale, W. H...Pittsburg, Pa.
Craven, Alfred. Kingsbridge, N. Y.
Creuzbaur, R. Walter
New York City
Crowell, Foster....New York City
Curtis, F. S....New Haven, Conn.

Davis, Charles....Allegheny, Pa.
Davis, Chandler...New York City

Davis, J. P..... New York City
Davis, J. Woodbridge
New York City
Davis, Robert B..Brooklyn, N. Y.
Dawley, Edwin P.
Providence, R. I.
Dean, Luther.....Taunton, Mass.
Deans, J. Sterling
Phœnixville, Pa.
Devin, George.....Chicago, Ill.
Deyo, S. L. F.....New York City
Dodge, R. D.....Brooklyn, N. Y.
Dunham, H. F....New York City
Duryea, Edwin, Jr.
Brooklyn, N. Y.

Ellis, John W..Woonsocket, R. I.
Erlandsen, Oscar..Jamaica, N. Y.
Evans, M. E.... .New York City

Farley, John M.
White Plains, N. Y.
Farnum, H. H....New York City
Farrington, H.
Croton-on-Hudson, N. Y.
Fehr, H. R.....Easton, Pa.
Ferris, F. E....Jersey City, N. J.
Field, William P...Newark, N. J.
Fisher, F. D.....Brooklyn, N. Y.
FitzGerald, Desmond
Boston, Mass.
Fouquet, J. D.....New York City
Francis, Henry N. Providence, R. I.
Freeman, E. G....New York City
Freeman, John R.
Providence, R. I.
French, Alexis H.
Brookline, Mass.
Frick, Walter....Carbondale, Pa.
Fteley, A.....New York City
Fuertes, J. H.....New York City

Gahagan, W. H...New York City
Gartensteig, Charles
New York City
Gay, Martin .....New York City
Geddes, J. K.....Zanesville, O.
Giles, Robert.....New York City
Goodnough, X. H..Boston, Mass.
Goodrich, W. F....Boston, Mass.
Gormly, W. B.....New York City
Gowen, Charles S.
Sing Sing, N. Y.
Graham, C. H.....New York City
Grant, T. H.....Red Bank, N. J.
Graves, Edwin D. Hartford, Conn.
Gray, William.....New York City
Gregory, C. E.....New York City
Green, Howard B..Buffalo, N. Y.
Greene, G. S., Jr..New York City
Greene, Robert M.
East Orange, N. J.

Haight, S. S.....New York City
Haines, H. S.....New York City
Haines, C. W.....Puebla, Mex.
Hankinson, A. W..New York City
Hansel, Charles....New York City
Harding, W. S..Philadelphia, Pa.
Harris, C. M.....New York City
Haskins, William J.
New York City
Hatton T. Chalkley
Wilmington, Del.
Haven, W. A.....Buffalo, N. Y.
Hayes, H. W....Fitchburg, Mass.
Hazen, Allen.....New York City
Hemming, D. W..New York City
Herbert, H. M.
Bound Brook, N. J.
Hering, Rudolph..New York City
Herschel, Clemens. New York City
Hickok, H. A.....Newark, N. J.
Hill, George.....New York City
Hill, W. R.....Syracuse, N. Y.
Hills, F. L.....Wilmington, Del.
Hoag, S. W., Jr...New York City
Hodgdon, F. W....Boston, Mass.
Horton, Thomas O. New York City
Howe, E. W.....Boston, Mass.

Hoxie, R. L...Washington, D. C.
Hoyt, J. T. N.....New York City
Hoyt, W. E.....Rochester, N. Y.
Humphrey, Richard L.
Philadelphia, Pa.
Hunt, Charles Warren
New York City
Hutton, W. R.....New York City

Ingersoll, C. M., Jr.Boston,Mass.
Irving, Walter E..New York City
Ives, Arthur S....Brooklyn, N. Y.

Johnson, W. C.
Niagara Falls, N. Y.
Just, George A....New York City

Kastl, Alex. E.....Clinton, Mass.
Kelley, W. D.....New York City
Kelly, C. W....New Haven, Conn.
Khuen, Richard, Jr. Pencoyd, Pa.
Kimball, George A.Boston, Mass.
King, Paul S.....New York City
Knap, J. M.....New York City
Knapp, Louis H....Buffalo, N. Y.
Knowles, Morris...Pittsburg, Pa.

Lawton, William H.Newport, R. I.
Leavitt, Charles W., Jr.
New York City
Leffingwell, F. D..New York City
Leisen, Theodore A.
Wilmington, Del.
Leonard, H. R..Philadelphia, Pa.
Lesley, R. W...Philadelphia, Pa.
Lewinson, M.....New York City
Lewis, N. P.....Brooklyn, N. Y.
Lieb, J. W., Jr....New York City
Lindenthal, D.....New York City
Lindenthal, G.....New York City
Linton, Harvey.....Altoona, Pa.
Locke, F. B..North Adams, Mass.
Long, Thomas J...New York City
Looker, H. B..Washington, D. C.
Loomis, Horace...New York City
Low, George E.Maplewood, N. J.
Lowinson, O.....New York City
Lundie, John.....Chicago, Ill.

MacGregor, R. A..New York City
McCann, Thomas H.
Hoboken, N. J.
McKenzie, T. H.
Southington, Conn.
McKim, Alex. Rice
New York City
McMinn, T. J.....New York City
Macdonald, Charles
New York City
Maclay, W. W....New York City
Marstrand, O. J....New York City
Martin, C. C.....Brooklyn, N. Y.
Martin, Kingsley L.
New York City
Martin, W. B.....Newark, N. J.
Mayer, Joseph....New York City
Melvin, D. N.Linoleumville, N. Y.
Merriman, Mansfield
South Bethlehem, Pa.
Meyer, Henry C...New York City
Meyer, Henry C., Jr.
New York City
Miller, A. M...Washington, D. C.
Miller, H. A.....Clinton, Mass.
Miller, Rudolph P.New York City
Montony, L. G....New York City
Moore, W. H..New Haven, Conn.
Morison, George S.
New York City.
Morse, C. A..Fort Madison, Iowa.
Myers, C. H.....New York City
Myers, John H., Jr.
Brooklyn, N. Y.

Neumeyer, R. E..Bethlehem, Pa.
Nichols, C. H..New Haven, Conn.
Nichols, O. F.....Brooklyn, N. Y.
North, Edward P..New York City

Olney, L. F.....New York City

Opdyke, Stacy B., Jr.
Philadelphia, Pa
O'Rourke, J. F....New York City
Owen, James.....Newark, N. J.

Parker, A. McC....New York City
Parsons, H. de B..New York City
Paschke, Theodore
Saugerties, N. Y.
Pegram, George H.New York City
Perkins, P. S.....Boston, Mass.
Peterson, P. Alex. Montreal, Que.
Pierce, W. T.....Boston, Mass.
Plympton, George W.
Brooklyn, N. Y.
Porter, J. M .....Easton, Pa.
Prichard, H. S.....Trenton, N. J.
Prout, H. G.....New York City
Pruyn, F. L. .....New York City

Rafter, George W.
Rochester, N. Y.
Ransome, Ernest L New York City
Raymond, William G..Troy, N. Y.
Reece, Benjamin
Philadelphia, Pa.
Rice, George S.....Boston, Mass.
Richardson, C. ....New York City
Richardson, S. F..Clinton, Mass.
Ricketts, Palmer C....Troy, N. Y.
Ridgway, R.....New York City
Robinson, A. W.
South Milwaukee, Wis.
Rogge, J. C. L ....New York City
Rood, H. M Mount Vernon, N. Y.
Rosenberg, F . ...New York City
Rotch, William.....Boston, Mass.
Rousseau, H. H .....Troy, N. Y.
Rowell, George F.Brooklyn, N. Y.
Rowland, Thomas F., Jr.
Brooklyn, N. Y.
Ruddle, John..Mauch Chunk, Pa.

Schneider, C. C.....Pencoyd, Pa.
Scott, A. M...Charleston, W. Va.
Seaman, Henry B..New York City
See, Horace.....New York City
Sherrerd, M. R.....Newark, N. J.
Shirreffs, R.... ...Boston, Mass.
Sibert, William L.
Willetspoint, N. Y.
Simpson, George F.New York City
Skinner, Frank W..New York City
Smith, Eugene R .....Islip, N. Y.
Smith, J. Waldo ..Paterson, N. J.
Smith, Oberlin..Bridgeton, N. J.
Smith, Wilson Fitch
New York City
Stearns, F. P .....Boston, Mass.
Stern, Eugene W..New York City
Stevens, Alexander.New York City
Stewart, J. M ....New York City
Stowe, H. C.....New York City
Strachan, Joseph.Brooklyn, N. Y.
Strobel, C. L.....Chicago, Ill.

Taber, George A...New York City
Taylor, Lucian A...Boston, Mass.
Taylor, W. J..Bound Brook, N. J.
Temple, W. H. G.
Providence, R. I.
Thacher, Edwin...Paterson, N. J.
Thayer, J. W....Scarsdale, N. Y.
Theban, J. G .....New York City
Thompson, S. C...New York City
Thomson, G. H....New York City
Thomson. John...New York City.
Tillson, George W.Brooklyn, N. Y.
Tomkins, C .....New York City
Tompkins, E. de V.New York City
Tompson, G. M..Wakefield, Mass.
Towle, Stevenson..New York City
Trautwine, John C., Jr.
Philadelphia, Pa.
Treadwell, Lee....New York City
Tribus, L. L .....New York City
Triest, W. G.....New York City
Trotter, A. W.....New York City
Turner, E. K .....Boston, Mass.
Tuttle, A. S .....Brooklyn, N. Y.

Ulrich, D.....New York City

Van Winkle, E. B..New York City
Vedeler, G. H.....New York City
Vielé, M. A.....Wyncote, Pa.
Vorce, C. B.....Hartford, Conn.

Wagner, S. T...Philadelphia, Pa.
Walker, C. I.....New York City
Ward, C. D.....New York City
Ward, C. R .....New York City
Ward, J. F.....New York City
Ward, L. B.....Jersey City, N. J.
Washington, W. DeH.
New York City
Watkins, F. W.
Mount Kisco, N. Y,
Webster, A. L.....New York City
Webster, G. S..Philadelphia, Pa.
Wegmann, E.....Katonah, N. Y.
Wells, Joseph A...New York City
Welton, B. F.....New York City
Wheeler, H. R..Philadelphia, Pa.
Whinery, S.....New York City
White, H. F New York City
Whitney, Frank O..Boston, Mass.
Wilcox, F. E.....New York City
Wiley, William H..New York City
Wilgus, W. J.. .. New York City
Wilson, C. W. S.
New Rochelle, N. Y.
Wisner, George Y..Detroit, Mich.
Wood, H. B .....Boston, Mass.
Wright, J. Bodine.New York City

York, H. W.....New York City

MINUTES OF MEETINGS.

OF THE SOCIETY.

February 1st, 1899.—The meeting was called to order at 20.30 o'clock; James Owen, Director, in the chair; Charles Warren Hunt, Secretary, and present, also, 56 members and 13 visitors.

The minutes of the meeting of January 18th, 1899, as printed in *Proceedings* for January, 1899, were accepted, and the following correction ordered to be made: On page 4, the first paragraph, reading "E. L. Corthell, M. Am. Soc. C. E., addressed the meeting on the 'Establishment of an International Institute of Engineers,'" should read "E. L. Corthell, M. Am. Soc. C. E., addressed the meeting on the 'Establishment of an International Congress of Engineering.'" A similar correction should be applied to the third paragraph on the same page.

A paper by John W. Hill, M. Am. Soc. C. E., entitled "The Accuracy and Durability of Water Meters," was presented by the Secretary, who also presented written communications on the subject from Messrs. James H. Harlow and W. C. Hawley.

The paper was discussed orally by Messrs. John Thomson, J. Waldo Smith and Clemens Herschel, and illustrated with stereopticon views.

Ballots were canvassed and the following candidates declared elected.

As Members.

John Charles Churchill, Jr., Oswego, N. Y.
Thomas Amory Coffin, New York City.
George William Frank, New York City.
Edward Macaulay Hartrick, Galveston, Tex.
George Hill, New York City.
Frank McMillan Stanton, Atlantic Mine, Mich.
Ellis Dunn Thompson, Philadelphia, Pa.

As Associate Members.

Frederic William Andros, Chihuahua, Mex.
Edward James Beugler, Bridgeport, Conn.
Clinton Sumner Burns, Kansas City, Mo.
Theodore Lincoln Condron, Chicago, Ill.
John Thompson Eastwood, New Orleans, La.
Thomas J. Foster, New York City.
George Rockwell Putnam, Washington, D. C.
Nicholas Doremus Wortendyke, Jersey City, N. J.

The Secretary announced the election by the Board of Direction on January 31st, 1899, of the following candidates:

As Associates.

Clarence Marvin Foster, Chicago, Ill.
Lawrence Cowle Phipps, Pittsburg, Pa.

As Juniors.

George Corrie Bartram, Boston, Mass.
Frederick Edwards, Hudson, N. Y.
James Peter Whiskeman, New York City.

The Secretary announced the death of the following members: Winthrop Bartlett, elected Member, January 2d, 1889; died January 16th, 1899. George Sears Greene, elected Member, November 5th, 1852; Honorary Member, October 26th, 1888; President from November 3d, 1875, to November 7th, 1877; died January 28th, 1899.

The Secretary announced that a special meeting would be held at 20.30 o'clock on Friday, February 10th, 1899, for an informal discussion on "Dry-Docks—Stone *vs.* Wood."

Adjourned.

February 10th, 1899. Special Meeting.—The meeting was called to order at 20.45 o'clock, Edward P. North, Vice-President, Am. Soc. C. E., in the chair; Charles Warren Hunt, Secretary, and present, also, 80 members and 19 visitors.

An informal discussion* on "**Dry-Docks—Stone *vs.* Wood**" was opened by William L. Cathcart, late Chief Engineer, U. S. N., Professor of Marine Engineering at the Academy of Naval Architecture, New York. The subject was discussed by Commodore George W. Melville, Engineer-in-Chief, Bureau of Steam Engineering, U. S. N.; Commodore M. T. Endicott, M. Am. Soc. C. E., Chief, Bureau of Yards and Docks, U. S. N.; P. C. Asserson, M. Am. Soc. C. E., Civil Engineer, U. S. N.; John Kennedy, M. Am. Soc. C. E., Chief Engineer, Harbor Commissioners, Montreal, Canada; R. G. Packard, M. Am. Soc. C. E.; J. N. Greene, M. Am. Soc. C. E.; Emil Diebitsch, Assoc. M. Am. Soc. C. E.; Oscar Lowinson, Jun. Am. Soc. C. E.; W. F. Whittemore, Assoc. M. Am. Soc. C. E.; Thomas D. Pitts, Jun. Am. Soc. C. E.; J. G. Tait, Assoc. M. Am. Soc. C. E.; E. L. Corthell, M. Am. Soc. C. E.; James Ritchie, M. Am. Soc. C. E.; H. W. Brinckerhoff, M. Am. Soc. C. E., and George S. Greene, Jr., M. Am. Soc. C. E.

Adjourned at 23.30 o'clock.

* This discussion will be printed in *Proceedings* as soon as possible.

February 15th, 1899.—The meeting was called to order at 20.45 o'clock, Edward P. North, Vice-President, in the chair; Charles Warren Hunt, Secretary, and present also 67 members and 11 guests.

The paper for the evening, entitled "The Approaches and Transportation Facilities of the Paris Exposition of 1900," by E. L. Corthell, M. Am. Soc. C. E., was presented by the author, and illustrated with stereopticon views.

The subject was discussed by Mr. John Goetz, Superintendent of the Department of Decorations, U. S. Commission for the Paris Exposition of 1900.

Adjourned.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society is open every day, except Sunday, from 9 to 22 o'clock.

MEETINGS.

Wednesday, March 1st, 1899, at 20.30 o'clock, a regular meeting will be held, at which a paper by Robert P. Woods, Jun. Am. Soc. C. E., entitled "Street Grades and Cross-Sections in Asphalt and Cement," will be presented. It is printed in this number of *Proceedings*.

Wednesday, March 15th, 1899, at 20.30 o'clock, a regular meeting will be held.

No paper has been presented for this meeting, but there will be an informal discussion, on a subject to be announced later.

DISCUSSIONS.

Discussion on the paper by J. W. Hill, M. Am. Soc. C. E., entitled "Accuracy and Durability of Water Meters," which was presented at the meeting of February 1st, 1899, will be closed March 15th, 1899.

Discussion on the paper by E. L. Corthell, M. Am. Soc. C. E., entitled "Approaches and Transportation Facilities at the Paris Exposition of 1900," which was presented at the meeting of February 15th, 1899, will be closed April 1st, 1899.

BOOK NOTICES.

THE TRANSITION CURVE, BY OFFSETS AND BY DEFLECTION ANGLES.

By C. L. Crandall, M. Am. Soc. C. E. Second edition, revised and enlarged. Morocco, 7 x 4 ins., 99 pp. New York, John Wiley & Sons, 1899.

The preface says: "Accurate methods are here, it is believed, for the first time, developed for the true transition curve, curvature increasing directly with the distance, which will hold for large central angles for both the offset and the deflection methods.
* * * * * * * * * * * * * *

"The offset method is worked up for use during location and the greater part of construction on account of its extreme simplicity."

On account of the increased use of transition curves for electric railroads, Table 1, containing offsets and other data, has been extended to circular curves of short radii. Some of the other tables have also been extended.

Eight pages are devoted to the theory of transition curves and thirty-one to practice; there are eight tables of offsets, coördinates, angles, radii, etc.

MATTER, ENERGY, FORCE AND WORK.

A plain presentation of fundamental physical concepts and of the Vortex-Atom, and other theories. By Silas W. Holman. Cloth, 8 x 5 ins., 257 pp., $2.50. New York. The Macmillan Company, 1898.

The work is divided into two parts. Part one contains the subject-matter proper. The definitions and views given are not designed to be an exposition of current and accepted usage, but an attempt at a setting forth of individual thought.

Part two consists of summaries of the chief theories of the nature of matter, energy and force.

Heading of chapters are: Substance, Matter; Motion; Energy; Forms of Energy; Force; Kinetic Energy, Kinergety; Force Measurement; Work; Potential Energy; Matter; Comments on Certain Definitions; Function of Theory and Hypothesis; Kinetic Theory of Gases; Le Sage's Theory of Gravitation; The Vortex-Atom Theory; Nature of Energy and Matter. The book closes with an index.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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STREET GRADES AND CROSS-SECTIONS IN ASPHALT AND CEMENT.

By ROBERT P. WOODS, Jun. Am. Soc. C. E.

TO BE PRESENTED MARCH 1ST, 1899.

Certain rules and practices in street improvements have been formulated in the larger cities, which, although desirable, cannot always be applied to similar work in the smaller ones. The size, topography, and general local conditions of a city largely determine the character of the street grades and cross-sections, and their plan of arrangement.

A slight departure from customary methods was made by the author in arranging the grades and cross-sections to be used in the recent construction of sheet-asphalt roadways and cement walks, curbs and gutters, in a part of the business district of Wabash, Ind., a small city, having a population of about 11 000. The old grades, as

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. The papers with discussion in full will be published in the volumes of *Transactions*.

established, were poorly adapted for the proposed improvement, and a careful adjustment of those involved was prepared, with the object of having the finished work as substantial, convenient for use, and harmonious as the local circumstances would permit. Generally, the improvement might be regarded as unnecessary side-hill construction on a comparatively flat surface; a condition unwarranted in view of the original topography of the included territory, and yet not now admitting of radical changes.

The erection of many of the buildings within the limits of the improvement had preceded the adoption of any grades by the city authorities, and to this tardiness of the proper municipal officers in establishing common gradients, may be attributed much of the irregularity in the elevations of the floors of the different business rooms.

In the rearrangement of grades, the sidewalks were raised from 1 in. to 12 ins. above the floors of some dozen adjoining store rooms. The owners of the buildings affected, in several instances, waived all claims for damages occasioned by such changes, and the remaining cases were referred to the Board of City Commissioners for assessment of benefits and damages.

There are few places in the improvement where the two opposite walks on any street are on the same grade. This inequality between the elevations of opposite points on the property lines of any street attains a maximum of 2.45 ft. on a street width of 90 ft., and 1.65 ft. on a street width of 66 ft.

The prominent feature in the arrangement of the grades and sections described in this paper is what is here called the "double curb," which consisted of a lower or roadway section and an upper or sidewalk section, so shaped as to form steps from the gutter to the walk. The principal factors entering into the details of the adjustment were the determination of the location of the crown line of the roadway, the height of the exposed vertical face of the curb, and the form and amount of transverse slope of the roadway and walk.

The streets (Fig. 1) are either 90 ft. or 66 ft. wide between outside or property lines, and are paved for the full width with asphalt or cement, with the exception of one block on Market street (not shown in Fig. 1), where half the sidewalk space on either side of the roadway

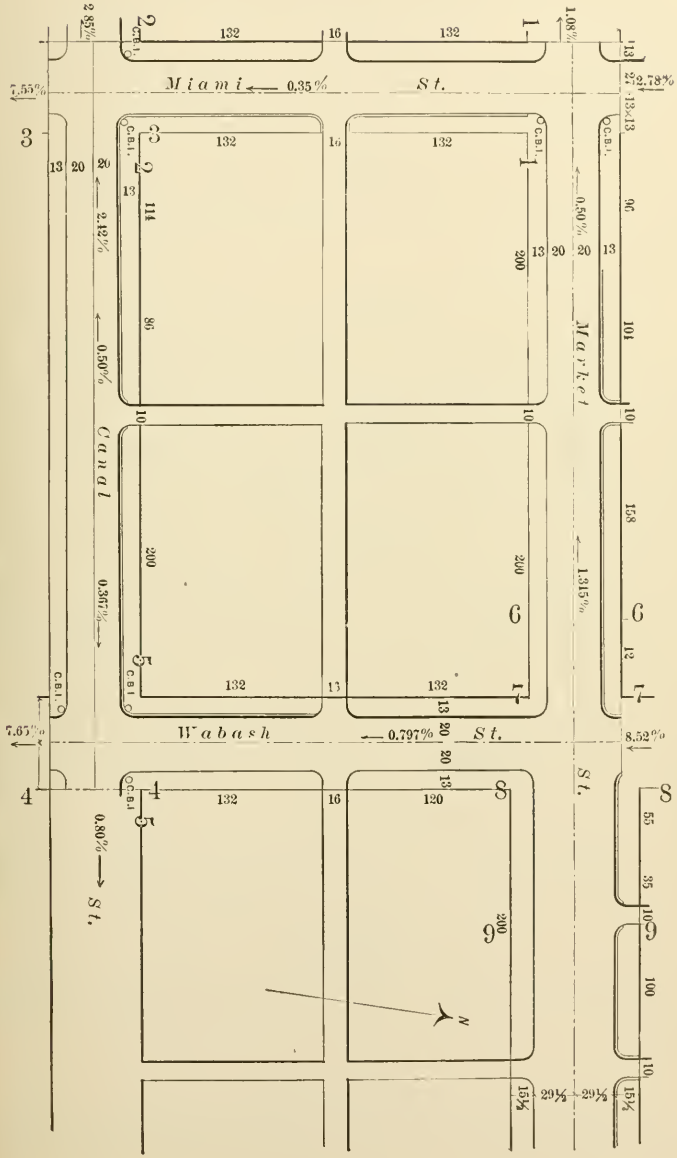

Miami St.
0.35%
Market
0.50%
Canal
2.42%
0.50%
0.36%
Wabash St.
0.79%
St.
0.80%
St.
1.31%
2.85%
7.55%
1.08%
2.78%
7.65%
8.52%
C.B.I.
N

Fig. 1.

is paved with cement and the other half graded into lawn. On the 90-ft. street, the roadway between curb faces is 59 ft., and either walk, including the curb, is 15½ ft. wide. The 66-ft. streets have roadways 40 ft. in width between curb faces, and either walk, including the curb, is 13 ft. wide.

All roadways, exclusive of gutters, are paved with sheet asphalt consisting of a 2-in. top, and a 1-in. bituminous binder, on a 6-in. hydraulic concrete base. The walks, curbs and gutters are laid with granite-faced cement concrete; the walks being 5 ins. and the curbs and gutter 6 ins. thick.

The curb and gutter throughout were combined, and constructed as a monolith, in sections 6 ft. in length. The roadway edge of the curb is turned with a 2-in. radius, and the junction of the curb and gutter was effected with a radius of 1½ ins. From out to out the combined curb and gutter measures 22 ins.; 6 ins. for the curb and 16 ins. for the gutter. The exposed vertical height of the curb varies from a minimum of 0.33 ft. to an extreme of 0.85 ft.

The double curb is composed of a lower or roadway section and an upper or sidewalk section constructed on parallel lines; the roadway face of the latter being 12 ins. from the roadway face of the roadway curb and the top flush with the sidewalk. The exposed vertical height varies from nothing to a maximum of 0.92 ft., and the edge is turned with a 2-in. radius. The lower or roadway section has a top width of 12 ins., and is the regular roadway curb increased by the addition of a 6-in. block of cement fitted into the space between the regular curb and the sidewalk curb, the top surface of the additional piece being flush with the top of the roadway curb. The exposed vertical height of the roadway section varies, as in the case of the regular roadway curb, from 0.33 ft. to 0.85 ft. This double curb was designed to be used where the height from the gutter to the top of the walk was more than 10 ins., but did not exceed 21 ins. Risers of 10 and 11 ins. in steps are rather excessive, but it was thought that the exigencies of the situation permitted their adoption in the few places where they were used.

All cut-off boxes for gas and water supply to buildings, were placed in the sidewalk space immediately back of the regular curb, and where encountered contiguous to the double curb, they were fitted into the widened portion of the lower section.

OUTLINES OF CROSS SECTIONS OF STREETS.

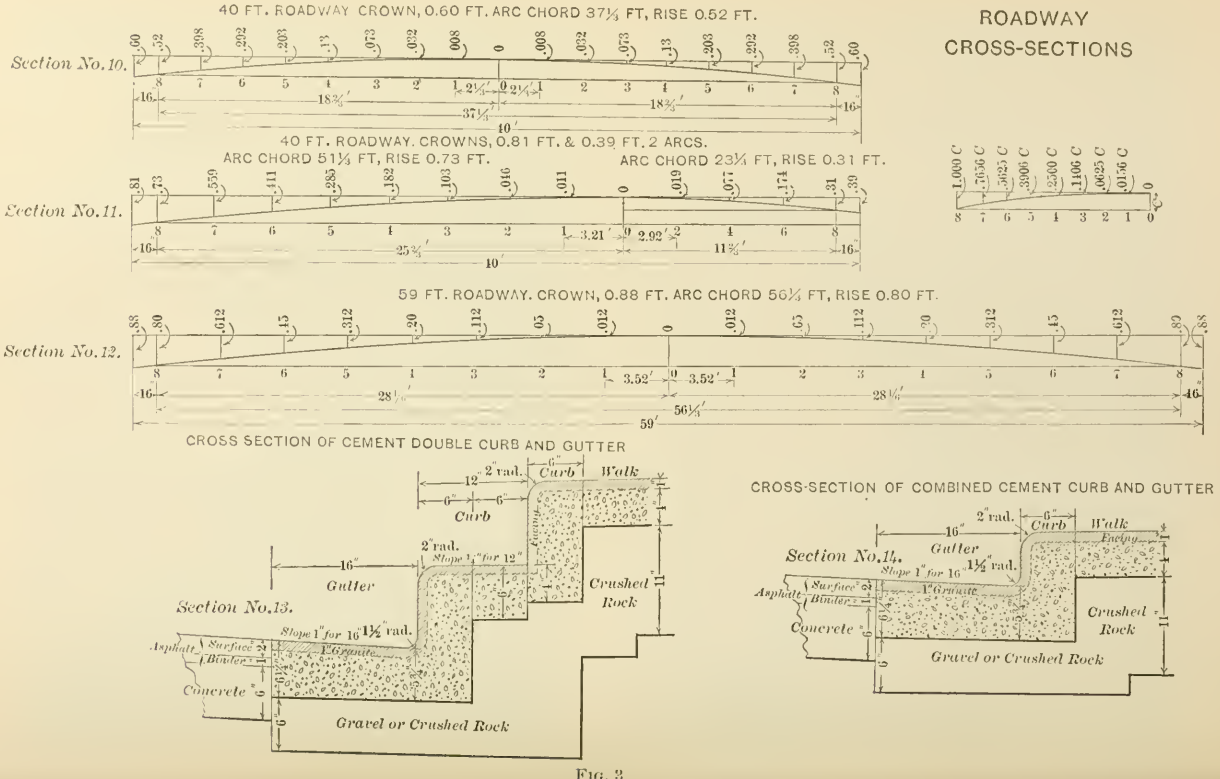
ALL MEASUREMENTS IN FEET.

FIG. 2.

Several short stretches of good stone sidewalks had been constructed by property owners under previous city ordinances, and after some adjustment were allowed to remain. They comprised the north walk on Canal Street, from Miami Street to Wabash Street, and several small lengths on both sides of Market Street, between Wabash and Huntington Streets. Those on the north side of Market Street had been paved to a width of only 12 ft., thus necessitating a widening of several feet.

Frequently, in street construction, where the side lines are on different gradients, one gutter is placed higher than the other, and the transverse form of the roadway made either on a straight line between the two gutters, or in the form of half an arc, the chord of which is double the width of the roadway. When the former is used, sags or depressions have a tendency to develop in the pavement, especially where the roadway is wide, and with either one, the objection might be raised, that carriage and street car travel are not best accommodated. It was decided to preserve a crown line within the roadway throughout this improvement, due consideration having been given to other conditions; and to keep it preferably in the center; but, as an expedient, in one block of pavement on Miami Street, the crown line was shifted 7 ft. from the center toward the high side of the street, as shown in Sections Nos. 1 and 2, Fig. 2. With this single exception the crown line was maintained in the center of each roadway.

The three usual forms for roadway crowns are, the parabolic and circular curves, or a combination of either one with tangent sides. Of these, the first two are nearly always alike, while the last may be arranged quite differently from either. The form used by the author was a parabolic curve, and yet, for all practical purposes, it might be called the segment of a circle, since a calculation of both curves shows their corresponding ordinates to be almost identical where the proportion of rise to span is so small as prevails in the construction of city pavements. The distance from the edge of the gutter to the crown line was divided into eight equal parts, the division points being numbered consecutively from "0" at the crown line, to "8" at the gutter, and the formula, as deduced for determining the ordinates from the surface of the pavement to a horizontal line level with the crown point, is as follows: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Section No.11.
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Section No.12.
CROSS SECTION OF CEMENT DOUBLE CURB AND GUTTER
Section No.13.
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Surface
Binder
Concrete
Gravel or Crushed Rock
Crushed Rock
Gutter
Curb
Walk
Grade
CROSS-SECTION OF COMBINED CEMENT CURB AND GUTTER
Section No.14.

FIG. 2.

Ordinate at the crown point 0 = 0

" " 1 = 0.0156 C or $\frac{1}{64}$ C

" " 2 = 0.0625 C " $\frac{4}{64}$ C

" " 3 = 0.1406 C " $\frac{9}{64}$ C

" " 4 = 0.2500 C " $\frac{16}{64}$ C

" " 5 = 0.3906 C " $\frac{25}{64}$ C

" " 6 = 0.5625 C " $\frac{36}{64}$ C

" " 7 = 0.7656 C " $\frac{49}{64}$ C

" edge of gutter " 8 = 1.0000 C " $\frac{64}{64}$ C

in which C equals the crown, for the distance specified.

The amount of crown of each roadway from the face of the curb to the crown line, is 3% of the horizontal distance between the two points named. Of this amount the gutter has a uniform slope of 0.08 ft., in its width of 16 ins., and the remainder is the rise or crown for the asphalt portion. In the case of a roadway 40 ft. in width between curb faces, with the crown line in the center, the total rise or crown is 0.60 ft., and the crown in the asphalt portion which is included between the gutter edges is 0.52 ft. This latter rise, as before stated, is proportioned on the form of a parabolic curve.

The crown and gutter lines in each block of pavement are on parallel longitudinal grades throughout, excepting the small deviations made on Market Street on either side of the Wabash and Market Street intersection. These roadway grades range from a minimum of 0.35% to a maximum of 2.42 per cent.

The cross-slope of the sidewalks is on a uniform inclination from the property lines of the streets to the curbs, and varies in amount from .015% to an occasional 4% grade.

The plan, Fig. 1, exhibits the dimensions of the streets, the location of the regular and double curbs and the longitudinal grade of the roadways. The two large similar figures on the same line refer to the numbers of the detailed outline sections shown in Fig. 2.

Outlines of nine different cross-sections of the streets are represented in Fig. 2, indicating the amount of slope given to the roadway and walks, together with the excess of elevation of one walk over the other at the location cited. Sections Nos. 10, 11 and 12, in Fig. 3, are the forms of crown for the several roadways, with the ordinates given in decimals of a foot, as calculated by the parabolic formula. The form for the 40-ft. roadway having the crown line in the center, is

FIG. 1.

FIG. 2.

shown in section No. 10, the curve corresponding closely with a circular arc having a radius of 335.30 ft. Section No. 11 is the form for the 40-ft. roadway having the crown line shifted 7 ft. from the center, and contains two curves. The short one on the right has a rise or middle ordinate of 0.31 ft., a half chord of $11\frac{2}{3}$ ft., and corresponds closely with a circular arc having a radius of 219.68 ft.; the longer one on the left has a rise or middle ordinate of 0.73 ft., a half chord of $25\frac{2}{3}$ ft., and corresponds with a circular arc having a radius of 451.58 ft. The form shown in section No. 12, was used for the 59-ft. roadway, the corresponding circular arc having a radius of 496.25 ft.

The double curb and gutter, and the combined single curb and gutter are shown in sections Nos. 13 and 14, respectively.

Two views of Wabash Street, taken before and after the construction of the improved pavement, are shown on Plate V.

HISTORICAL SKETCH

OF THE

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By Charles Warren Hunt, M. Am. Soc. C. E.

Cloth, 6 x 9 Inches.

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At the Annual Meeting, January 19th, 1898, the following facts in regard to the subscription to this book were brought out:

Two thousand copies were printed; 300 were bound in full morocco, of which 216 have been sold at $10 per copy, the resulting net profit being $943.06. Seventeen hundred copies, which have been paid for, are still on hand, and the Board of Direction was requested to consider the propriety of offering to the membership these copies bound in a less expensive style and at a reduced price, the net proceeds to be applied to the building fund.

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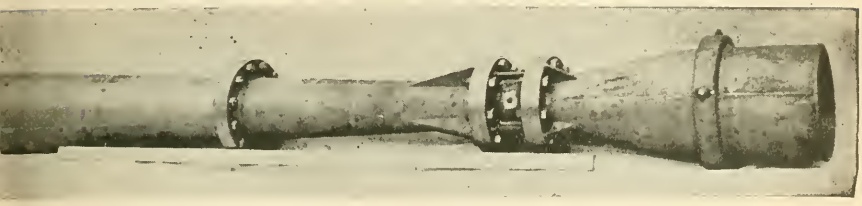

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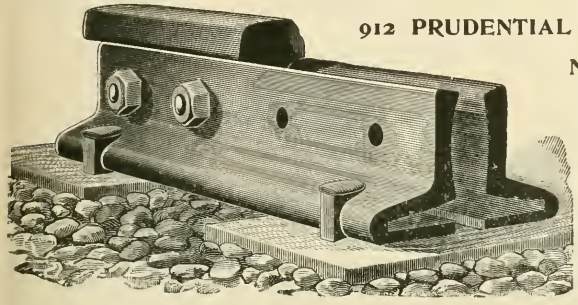

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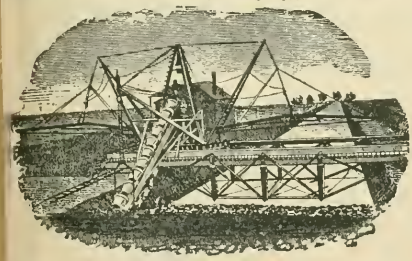

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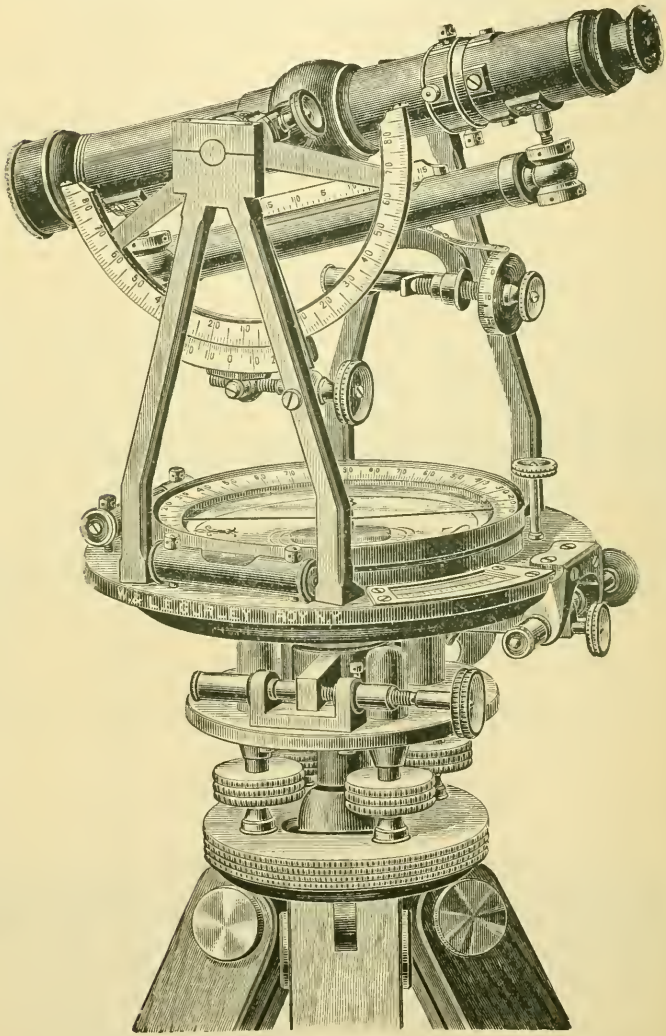

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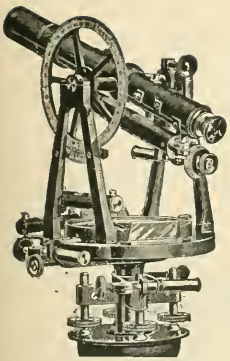

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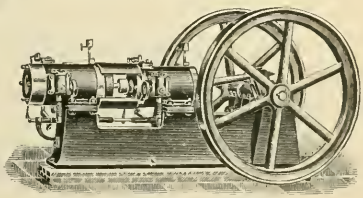

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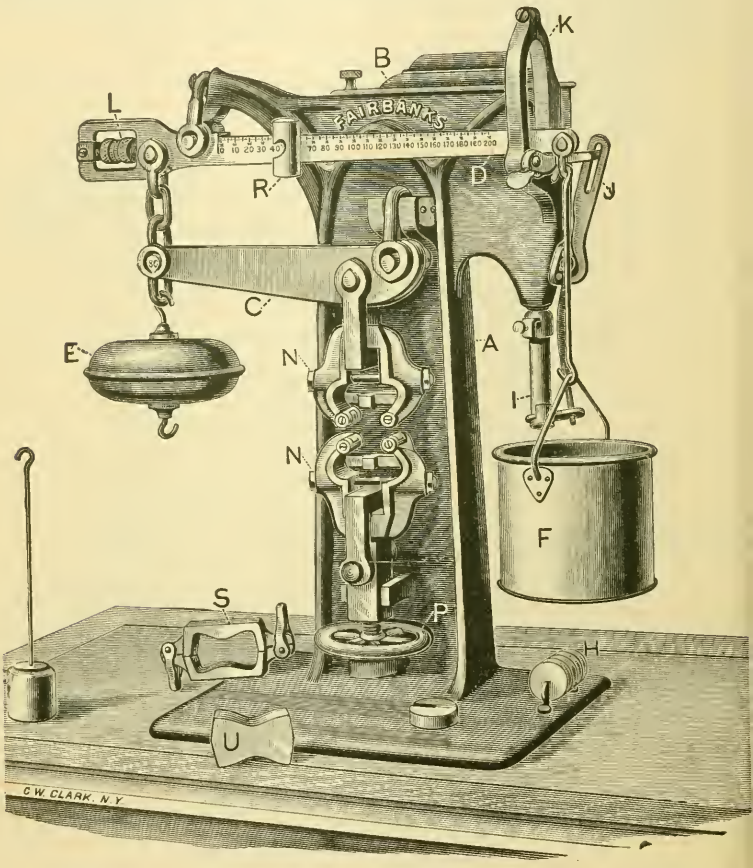

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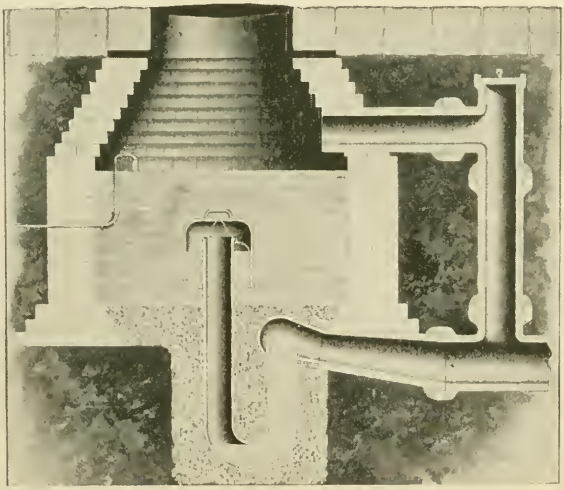

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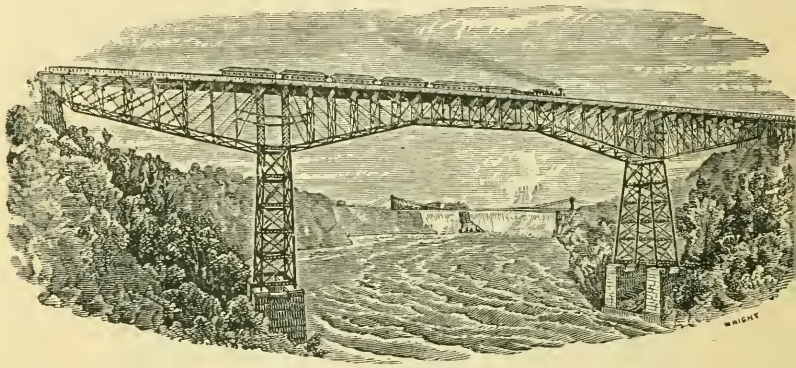

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OF

CIVIL ENGINEERS

March, 1899

PROCEEDINGS = VOL. XXV—No. 3

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Vol. XXV. MARCH, 1899. No. 3.

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publications.

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The prices of publications are as follows: Proceedings, $6 per annum; Transactions, $10 per annum. Postage will be added when they are sent to foreign countries.

American Society of Civil Engineers.

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Special Committees.

ON UNIFORM STANDARD TIME:—Sandford Fleming, Charles Paine, Theodore N. Ely.

ON ANALYSIS OF IRON AND STEEL:—Sub-Committee of the American Society of Civil Engineers (of the International Committee on Standards for the Analysis of Iron and Steel, of which Prof. J. W. Langley is Chairman)—Charles B. Dudley, William Metcalf, Thomas Rodd, A. E. Hunt.

ON UNITS OF MEASUREMENT:—George M. Bond, William M. Black, R. E. McMath, Charles B. Dudley, Alexander C. Humphreys.

ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Noble George S. Webster, O. M. Carter, W. B. W. Howe, Louis C. Sabin, H. W. York.

The House of the Society is open every day, except Sunday, from 9 to 22 o'clock.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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MINUTES OF MEETINGS.

OF THE SOCIETY.

March 1st, 1899.—The meeting was called to order at 20.30 o'clock; Edward P. North, Vice-President, in the chair; Charles Warren Hunt, Secretary, and present, also, 69 members and 11 visitors.

The minutes of the meetings of February 1st, 10th and 15th, 1899, were approved as printed in *Proceedings* for February, 1899.

A paper by Robert P. Woods, Jun. Am. Soc. C. E., entitled "Street Grades and Cross-Sections in Asphalt and Cement," was presented by the Secretary, who also presented a written communication on the subject from P. W. Henry, Assoc. M. Am. Soc. C. E.

The paper was discussed orally by Messrs. A. W. Hankinson, G. W. Tillson, James Owen, D. Ulrich, J. M. Evans and Edward P. North.

Ballots were canvassed and the following candidates declared elected:

As Memeers.

Howard Judson Cole, New York City.
Charles Henry Cooke, New York City.
Arthur Bateman Corthell, Boston, Mass.
John Christian Ostrup, Boston, Mass.
Douglas William Ross, Boisé, Idaho.
Ralph Martin Shankland, Chicago, Ill.
William Dana Taylor, Baton Rouge, La.

As Associate Members.

Charles Albertson, Kobe, Japan.
Samuel Pond Baird, Portsmouth, Ohio.
William Parsons Boright, Massena Springs, N. Y.
Henry Lawrence Cleverdon, Grand Rapids, Mich.
Paul Davis Cunningham, Washington, D. C.
George Warren Fuller, Cincinnati, Ohio.
Clark Olds, Erie, Pa.
Nelson Bowman Sweitzer, Olympia, Wash.
Samuel Richards Thomas, Hokendauqua, Pa.

The Secretary announced the election by the Board of Direction on February 28th, 1899, of the following candidates:

As Juniors.

Arthur Benjamin Ilsley, New Haven, Conn.
Frank Sears Senior, Montgomery, N. Y.
Howard Flanders Taylor, Mena, Ark.

The Secretary announced that on Monday, March 13th, 1899, at 20.30 o'clock, a meeting of the Juniors of the Society would be held for the purpose of discussing, with the Committee on Publications, the proposition to hold meetings of the Juniors for the reading and discussion of papers.

Adjourned.

March 15th, 1899.—The meeting was called to order at 20.40 o'clock; Edward P. North in the chair; Charles Warren Hunt, Secretary, and present, also, 80 members and 19 guests.

Mr. Cope Whitehouse delivered a lecture on "The Reservoirs of the Nile," which was illustrated by stereopticon views.

The Secretary announced the death of Frederic Candee Weir, elected Member August 7th, 1872; died March 1st, 1899; and of Herbert Steward, elected Associate February 6th, 1894; died March 4th, 1899.

The Secretary announced a meeting of the Juniors of the Society, to be held April 12th, 1899, the subject for informal discussion being "The Duties and Responsibilities of an Inspector."

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

January 18th, 1899.—President FitzGerald in the chair; Chas. Warren Hunt, Secretary, and present, also, Messrs. Clarke, Cartwright, Bensel, Deyo, Fteley, Haines, Hering, North, Owen, Ricketts, Schneider, Stearns, Thomson, Turner, Whinery and Wisner.

The following Standing Committees were appointed:

Committee on Finance.—Samuel Whinery, *Chairman;* James Owen, S. L. F. Deyo, Robt. Cartwright, Henry Manley.

Committee on Publications.—Edward P. North, *Chairman;* John Thomson, Rudolph Hering, H. S. Haines, P. C. Ricketts.

Committee on Library.—Frederic P. Stearns, *Chairman;* J. A. Bensel, Chas. Warren Hunt, C. W. Buchholz, John Kennedy.

The Secretary then withdrew.

Chas. Warren Hunt was unanimously elected Secretary for the ensuing year.

The question of the salary of the Secretary was considered and referred to the Committee on Finance for report to the next meeting of the Board.

Mr. Hunt, having been advised of his re-election, returned and resumed his duties as Secretary.

It was resolved that the Society House be hereafter closed on Sunday.

The matter of providing for cross-discussion of papers was considered and referred to the Committee on Publications.

The President was authorized to appoint a special committee to report on the improvement of the acoustics of the Auditorium.

Adjourned.

January 31st, 1899.—President FitzGerald in the chair; Chas. Warren Hunt, Secretary, and present, also, Messrs. Bensel, Clarke, Deyo, Fteley, Hering, Morison, North, Owen, Schneider, Stearns, Thomson, Turner and Whinery.

The Committee on Finance reported, recommending that the salary of the Secretary be fixed at $6 000 per year.

It was moved and seconded that the salary of the Secretary be fixed at $6 000 per annum, and that the vote on this motion be taken by letter ballot of the whole Board. (Carried.)

The President was authorized to appoint a committee to report to the Board on the matter of the Paris Exposition of 1900.

The matter of holding meetings of Juniors was referred to the Committee on Publications for report.

The resignation of Wm. H. Brown, M. Am. Soc. C. E., was accepted.

Two candidates for Associate and three for Junior were elected.

Applications were considered and other routine business transacted.

Adjourned.

February 28th, 1899.—Vice-President North in the chair; Chas. Warren Hunt, Secretary, and present, also, Messrs. Bensel, Buchholz, Deyo, Hering, Owen, Ricketts, Thomson and Turner.

The letter ballot on the motion to fix the salary of the Secretary at $6 000 per annum was canvassed and declared carried by a vote of 21 to 7, the Secretary not voting.

A report from the Committee on Publications was received, recommending the adoption of certain rules for the Acceptance, Publication and Editing of Papers, Discussion and Correspondence.

The rules presented* were adopted.

A report was received from the Committee on Library, stating the progress made in re-classifying and indexing the Library, and asking for an appropriation of $800 for binding and for the purchase of books.

The request of the Committee was granted, and $800 appropriated for the purpose named.

A report was received from the Special Committee on the Paris Exposition of 1900, and the general subject considered at length.

The Committee of Arrangements for the Annual Meeting of 1899 made a final report.

The President was authorized to appoint a committee of the Board and a Local Committee of Arrangements for the Annual Convention.

Three candidates for Junior were elected.

Applications were considered and other routine business transacted.

Adjourned.

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* See p. 71.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society is open every day, except Sunday, from 9 to 22 o'clock.

MEETINGS.

Wednesday, April 5th, 1899, at 20.30 o'clock, a regular meeting will be held, at which a paper by L. M. Hoskins, Esq., entitled "General Criterion for Position of Loads Causing Maximum Stress in Any Member of a Bridge Truss," will be presented. It is printed in this number of *Proceedings*.

Wednesday, April 19, 1899, at 20.30 o'clock, a regular meeting will be held.

No paper has been scheduled for this meeting, but there will be an informal discussion on a subject to be announced later.

GENERAL RULES FOR THE ACCEPTANCE AND PRESENTATION OF PAPERS PRESENTED TO THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

Adopted by the Board of Direction, February 28th, 1899.

1. Papers offered to the Society should be sent to the Secretary. They shall be considered by the Committee on Publications, and, if accepted, be set down for presentation to the Society at one of its regular semi-monthly meetings. The publication of such papers shall be in accordance with the rules adopted by the Board of Direction governing the publication of Papers, Discussion and Correspondence.

2. No formal papers shall be set down for presentation to the Annual Convention, but in lieu thereof discussion will be asked for at each Annual Convention and at each Annual Meeting on all papers which have been published in *Proceedings* during the six months immediately preceding each of these general meetings.

3. Special subjects of engineering interest shall also be presented for discussion at each of these general meetings, and it shall be the duty of the Secretary to announce in the usual Convention or Annual Meeting circular the papers which, under this rule, will be open to additional discussion, together with such topics as may have been suggested to, and approved by, the Committee on Publications.

GENERAL RULES GOVERNING THE PUBLICATION OF PAPERS, DISCUSSION AND CORRESPONDENCE, BY THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

Adopted by the Board of Direction, February 28th, 1899.

1. Papers accepted by the Committee on Publications shall be printed promptly in *Proceedings* in advance of the date set for their presentation to the Society, with a foot-note inviting discussion and correspondence upon them.

2. Correspondence on any paper, which is received prior to the meeting at which the latter is read, shall be presented to that meeting, and shall be printed as soon as possible in a subsequent number of *Proceedings*.

3. Oral discussion shall be treated as indicated in Rule 7 of "Editorial Rules" and published in *Proceedings* as soon as possible.

4. The publication of Discussion and Correspondence in *Proceedings* shall be accompanied by a note stating that such publication is a preliminary one, subject to changes and corrections, and further correspondence upon the subject shall be invited.

5. The discussion on any paper shall be closed thirty days after the preliminary publication of the discussion and correspondence upon it, and the author may then prepare a closure, which shall be printed in a subsequent issue of *Proceedings*, and one month additional shall be allowed for further correspondence before the Paper, Discussion and Correspondence shall be sent to press for the next Volume of *Transactions*. Should any question arise as to the further continuance of any discussion, the whole matter shall be reported by the Secretary to the Committee on Publications, whose decision shall be final. In all cases the right of the Author to final closure of the discussion upon his paper shall be maintained.

EDITORIAL RULES FOR PUBLICATIONS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

Adopted by the Board of Direction, February 28th, 1899.

1. In preparing all Papers, Discussion and Correspondence on technical subjects for the press, the third person shall be used. The contributor of a paper shall be referred to as "the author," a correspondent as "the writer," and one who contributes to the verbal discussion as "the speaker."

2. All redundant matter, interrogations which the text of the paper renders needless, and sarcastic forms of expression, shall be excluded.

3. All errors in orthography and grammar shall be corrected. In cases where there are two correct spellings, that to which preference

is given by the best authorities shall, for the sake of uniformity, be adopted.

4. Ambiguous or obscure statements shall be restated in clearer form, care being taken, however, to preserve as far as possible the language and style of the author.

5. No unnecessary changes in wording shall be made.

6. Whenever time will permit, a proof of any Paper and Discussion in which any changes, indicated in Rules 4 and 5, have been made, shall be submitted to the author before publication. But in this connection it should be borne in mind that it is not always possible on account of lack of time to submit a proof and that inasmuch as the first publication of all discussions will be a preliminary one, necessary corrections may be made before the final issue in the Volumes of *Transactions*.

7. Oral discussions shall be reported stenographically. From this report a rearranged form of the matter brought out by the speaker shall be carefully prepared and forwarded to him, together with the full report of what he was understood to say, and, whenever time permits, any modifications or amplifications of this statement shall be printed in *Proceedings*. When it is not possible to secure the approval of the speaker prior to this publication, he shall be allowed to make corrections, changes or amplifications in his discussion before it is published in *Transactions*.

The resolution of the Board of Direction, adopting the foregoing rules, provides that they shall be published in *Proceedings*, with the reasons governing their adoption, in order that the whole matter may be fully understood by the Membership.

RULES FOR THE ACCEPTANCE AND PRESENTATION OF PAPERS.

The effect of these rules will be to change the existing system in the following particulars:

There will be no formal papers read at the Annual Convention, and in lieu thereof it is provided that there shall be a general discussion of all papers published in *Proceedings* during the six months immediately preceding the Convention.

It is also provided that a similar discussion shall be arranged for at the Annual Meeting, and that at both of these General Meetings timely topics of technical interest shall be announced for discussion.

The reasons governing the Board in the adoption of these changes, briefly stated, are:

1. Among the members of the Society the impression seems to prevail that papers presented to the Society at a Convention there attract more attention and receive more extended discussion than if presented at one of the regular semi-monthly meetings. This results in the withholding of papers until about the time of the Convention, and diminishes the number presented during that part of the year immediately preceding the month of June. The value of advance publication for the purpose of eliciting discussion is thereby diminished, it often being found impracticable to secure papers enough even in February and March to keep ahead of the regular meetings.

2. The assumption that wider discussion of papers is secured at conventions, is not borne out by experience. If, as has sometimes been the case, there are many papers scheduled for the Convention, it is difficult to allot time for a proper discussion of each of them, as it cannot be foretold how much interest each will excite. A member wishing to discuss one of these papers cannot tell just when it will be brought up, and he therefore must attend all sessions until it is considered, if he wishes to make sure of presenting his views. Cases have also been known, where, with a long programme and a limited time, discussions have been hurriedly closed, and proposed discussions inadvertently excluded.

3. It is believed that the throwing open for discussion at the Convention of all papers which have been presented to the Society during the preceding six months, and in addition providing timely topics of engineering interest for general and somewhat informal discussion, will result in adding life and interest to this General Meeting which is not possible when formal papers alone are presented. It should also be remembered, that the new rule in regard to prompt publication of discussion and correspondence will bring before the Convention, not only the paper itself, but the discussions to which it has already given rise.

4. The Annual Meeting has heretofore been devoted entirely to the Business of the Society, and the new rule will, it is believed, give to it additional professional interest.

RULES FOR THE PUBLICATION OF PAPERS, DISCUSSION AND CORRESPONDENCE.

Up to January, 1896, papers read before the Society were not published in advance to all Members of the Society. A few extra copies were printed which were sent to such Members as the Secretary thought might be willing to discuss them. After presentation to the Society, the paper was not published until all the discussion, includ-

ing the closing discussion of the author, had been collected. This consumed much time, and it was often six months, and sometimes longer, before the first publication of a paper. Under this system 80 or 90% of the membership never saw a paper until it was published with discussion. This was alike unfortunate for the author and for the membership in general.

Beginning with January, 1896, the system was changed. All papers were published in *Proceedings* in advance of the date set for presentation to the Society, thus giving all members an opportunity to take part in the discussion of them, either orally or by sending in a written communication, which is presented to the meeting.

There is no doubt that this system has worked well, but it has become evident that much additional interest would be added to our publications, if an arrangement could be made, whereby the discussion and correspondence could be promptly placed before all members, both for the purpose of keeping them in closer touch with the work of the Society, and to elicit discussion, not only of the paper, but also of the discussion itself.

A careful reading of these rules will show that the delay in final issue of papers in *Transactions* will not be material, and that the number of volumes of *Transactions* to be issued annually, as well as the date of such issue, will be dependent only on the volume of printed matter already published in *Proceedings*.

EDITORIAL RULES.

These rules do not change the methods now, and for some time past, in force in the editorial work. They have been formulated by the Publication Committee, and adopted and ordered printed by the Board of Direction, for the information of the membership.

It has been found by experience that the exclusive use of the third person, in papers and discussions, facilitates whatever editorial amendment, adaptation or condensation may be necessary or desirable, and if by its use there is any loss of individuality in particular cases, that loss is more than compensated by a uniformly impersonal statement of fact and opinion, which is of great benefit to the reader and student of engineering literature.

It is believed that the reasons governing the Board in the adoption of the balance of these rules will be evident without further comment.

PRELIMINARY PUBLICATION OF DISCUSSION AND CORRESPONDENCE.

Attention is called to the fact that, under the rules governing the publication of papers, discussion and correspondence, recently adopted

by the Board of Direction,* the first preliminary publication of discussions in *Proceedings* appears in this number.

The discussions thus published are: "Dry Docks: Stone *vs.* Wood," an informal discussion which took place at the meeting of February 10th, 1899; and the discussion on the paper by Robert P. Woods, Jun. Am. Soc. C. E., entitled "Street Grades and Cross-Sections in Asphalt and Cement," presented at the meeting of March 1st, 1899.

As stated elsewhere, the purpose of this preliminary publication is to bring out cross-discussion, and communications relating to either of these subjects are invited, which, if received prior to April 22d, 1899, will be published in a later number of *Proceedings;* and, subsequently, all such discussions will be published with the paper and the author's closure in *Transactions.*

CURRENT TECHNICAL LITERATURE.

In this number of *Proceedings* is printed for the first time, a list of engineering articles which have appeared in various weekly, monthly and quarterly technical publications during the past two months.

The list does not pretend to express any opinion as to the value of any of the articles mentioned, the object in its publication being merely to call to the attention of members such of the current literature as would appear to be of interest. Many members doubtless can find the articles referred to, in a local technical library, but the list is also intended to give sufficient information to enable one not so situated to procure any article promptly from its publisher.

It cannot be expected that such a list shall be complete, but the intention is, if found to be of sufficient interest to members, to continue it every month. There are many articles published serially, references to which would make the list too voluminous, and all such articles have been omitted.

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*See p. 71.

RECENT ENGINEERING ARTICLES OF INTEREST.

NOTE.—This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly.

The names and places of publication and, wherever possible, the price of each of the journals, are given in the following list with a Roman numeral prefixed. Wherever this numeral is found in the subjoined list of articles, it refers to the publication so indicated.

It is quite possible that this list, which it is intended to continue in each monthly number of *Proceedings* if sufficient interest therein is manifested by the membership, does not contain a reference to every interesting engineering article. There are many serials published in the technical press which it would be manifestly impossible to index from month to month, and all of these are omitted from this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Eng. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Eng., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Can.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Indicator*, Stevens Institute, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 30c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering*, W. H. Wiley, New York City, 35c.
(12) *The Engineer*, International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *Engineering Record*, New York City, 12c.
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City, 35c.
(18) *Railway and Engineering Review*, Chicago, Ill.
(19) *Scientific American Supplement*, New York City, 10c.

A Stadia Diagram. Morris K. Trumbull. (4) Dec., 1898.

The Plant of the Michigan Portland Cement Company, Coldwater, Mich. By Frederick H. Lewis, M. Am. Soc. C. E. (14) Feb. 25.
Comparative Tests of Different Forms of Cement Briquettes. Prof. Jerome Sondericker. (1) Jan., 1899.
Russian Portland Cement Specifications. (14) Feb. 18.

The New Masonry Dry Dock at Boston. (14) Feb. 4.
Masonry. George S. Morison, M. Am. Soc. C. E. (4) Dec., 1898.
The Theory and Design of the Masonry Arch. William H. Burr, M. Am. Soc. C. E. (6) Nov., 1898.

The New Croton Dam. (14) Jan., 7.
The Castlewood Rock-fill Dam and the Canal of the Denver Land and Water Company. W. P. Hardesty. (13) Feb. 9.

The Viaduct of Mungsten. (12) Jan. 20.
A Mélan Concrete Steel Railroad Bridge. (15) March 3.
Concrete and Expanded Metal Highway Bridge Construction in Allegheny Co., Pa. (13) Jan. 26.
The Old and New Suspension Bridges over the Niagara River at Lewiston, N. Y. (13) Jan. 12.
A Concrete Arch Bridge with Granite Hinges. (13) Feb. 16.
Standard Specifications for Steel. William R. Webster. (9) March, 1899.
The "Main Gut" Lift Span on the Northern and Western Railway, Newfoundland. W. Chase Thomson. (5) Vol. xii, Part i.
The Protection of Metal Work. (14) Jan. 7.
Tests of Bridge Paint. Ira O. Baker. (15) March 10.

Pennsylvania Avenue Subway and Tunnel. George S. Webster, M. Am. Soc. C. E. (2) Feb., 1899.
Passenger Lift Equipment for the Central London Railway. (11) March 3.
A New General Formula for Train Resistance. (17) Feb., 1899.
Office System Engineering Department of Elevated lines, Boston Elevated Railway Company. (14) Jan. 21.
Train Resistance. John Balch Blood. (17) March, 1899.
A New Train Resistance Formula. (17) Feb., 1899. (15) Feb. 17.
Formulas for Locating Frogs. Malverd A. Howe. (15) March 10.
Heavier Rails for Railroads Laid with Flange Rails, C. P. Sandberg. (15) Feb. 10.
Results Obtained in the Past Fifteen Years with Stiff and Heavy Rail Sections. P. H. Dudley. (13) March 2.
Elevation of the L. S. & M. S., and the C. R. I. & P. Railways. Marvin H. Dey. (4) Dec. 1898.
The Manning Improved Rail. W. T. Manning, M. Am. Soc. C. E. (15) Jan. 27.
The New Railroad Laboratory at Purdue University. (15) Jan. 20.
Track Elevation of the Pittsburg, Ft. Wayne & Chicago Railway. W. H. Coverdale. (4) Dec., 1898.
Track Elevation of the St. Charles Air Line. H. W. Parkhurst. (4) Dec., 1898.
Joint Track Elevation and Depression at Sixteenth and Clark Streets. G. W. Vaughn, M. Am. Soc. C. E. (4) Dec., 1898.

Sea Encroachments and Case Groynes. (11) Jan. 27.
Rock Blasting Under Water. (19) Jan. 7.
By-Pass Channel for the Chicago River. (13) Feb. 16.
Ten-Yard Clam-Shell Dredge for the Buffalo, N. Y., Breakwater Construction. (13) Feb. 2.

The Clarification of River Waters. Allen Hazen. Assoc. M. Am. Soc. C. E. (3) March, 1899.
Meters. W. C. Brough. (5) Vol. xii. Part i.
The Venturi Water Meter. Clemens Herschel, M. Am. Soc. C. E. (10) March, 1899.
Movement of Water Mains in connection with Pennsylvania Avenue Subway Work. Allen J. Fuller. (2) Feb., 1899.
The Reconstruction of the York, Pa., Water Supply. John Birkinbine. (2) Feb., 1899.
A Photographic Study of Fire Streams. (14) Feb. 18.
New Duluth Water-Works. (14) Jan. 21.
An Improved Filter for Microscopical Water Analysis. Daniel D. Jackson. (7) Dec., 1898.
The Berwyn, Pa., Sand Filter. (14) March 11.
The Hydraulic Laboratory of Cornell University. (13) March 2. (14) March 4.
The Hydraulic Laboratory at McGill. Prof. H. T. Bovey and J. T. Farmer. (5) Vol. xii, Part i.
Low-Water Measurements in the State of California During the Summer of 1898. J. B. Lippincott. (13) Jan. 12.
Further Experiments on the Character of Fluid Motion. Prof. H. S. Hele-Shaw. (12) Jan. 27.
Possibilities of Economy in Pumping Engines. (14) Jan. 28.
The Present Status of the Centrifugal Pump. Henry P. Jones. (13) March 9.
Duty Trial of a 30 000 000-Gallon Pump at Buffalo, N. Y. (14) March 4.

Sewage Disposal. Harry de B. Parsons. M. Am. Soc. C. E. (8) Jan., 1899.

Aerial Ropeway at London Portland Cement Company's Works. (11) Feb. 24.
The Production of Metallic Tubes by Extrusion. (11) Jan. 6. (16) March 4.
Structural Features of the Enlarged Standard Building, New York City. (14) Feb. 25.
Methods of Testing Paving Brick. (14) March 4.
Water Power Plants with Long Distance Electric Transmission in Southern California. C. E. Fowler, M. Am. Soc. C. E. (13) March 9.

LIST OF MEMBERS.

ADDITIONS.

MEMBERS.

			Date of Membership
Cole, Howard Judson	50 Wall St., New York City	Assoc. M.	Jan. 6, 1892
		M.	Mar. 1, 1899
Corthell, Arthur Bateman	220 South Station, Boston, Mass.		Mar. 1, 1899
Kirkpatrick, Walter Gill	City Engineer, Jackson, Miss.	Assoc. M.	April 6, 1892
		M.	Oct. 5, 1898
Ostrup, John Christian	Designing Eng., Boston Elevated Ry. Co., 101 Milk St., Boston, Mass.	Assoc. M.	Mar. 4, 1896
		M.	Mar. 1, 1899
Shankland, Ralph Martin	816 Rookery Bldg., Chicago, Ill.		Mar. 1, 1899
Taylor, William Dana	St. L., P. & N. R. R., Security Bldg., St. Louis, Mo.		Mar. 1, 1899

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Boright, William Parsons	34 West Congress St., care of U. S. D. W., Detroit, Mich.	Jun.	Jan. 2, 1894
		Assoc. M.	Mar. 1, 1899
De Witt, Elmore	Lock Box 183, Steelton, Pa.		Oct. 5, 1898
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Sweitzer, Nelson Bowman	Examiner of Surveys, General Land Office, Washington, D. C.	Jun.	Feb. 28, 1893
		Assoc. M.	Mar. 1, 1899
Thomas, Samuel Richards	M. M. and Eng., Ironton R. R., Hokendauqua, Pa	Jun.	Oct. 4, 1892
		Assoc. M.	Mar. 1, 1899

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Graham, William Woodman	Care of Messrs. Stalknecht, Durango, Mex.		Dec. 6, 1898

JUNIORS.

	Date of Membership.
Goodridge, John Wesley.....981 Greene Ave., Brooklyn, N. Y..... ...	Oct. 4, 1898
Ilsley, Arthur Benjamin.....Chf. Engr's. Office, N. Y., N. H. & H. R.R., New Haven, Conn.....	Feb. 28, 1899

CHANGES AND CORRECTIONS.

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Booth, William Henry.....Clevedon, Chatsworth Road, West Norwood, London, S. E., England.
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JUNIORS.

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DEATHS.

MICHIE, WILLIAM ROBERTS.....Elected Junior Feb. 28th, 1893; Associate Member Oct. 7th, 1896; died Feb. 2d, 1899.

STEWARD, HERBERT.....Elected Associate Feb. 6th, 1894; died March 4th, 1899.

WEIR, FREDERIC CANDEE.....Elected Member Aug. 7th, 1872; died March 1st, 1899.

BOOK NOTICES.

THE MICROSCOPY OF DRINKING WATER.

By George Chandler Whipple. First Edition. Cloth, 9 x 6 ins., 300 pp. New York, John Wiley & Sons, 1899.

This book is intended to serve as a guide to the water analyst and the water-works engineer, and to stimulate a greater interest in the study of microscopic aquatic life and general limnology, from the practical and economic standpoint. The work is elementary in character. Headings of chapters in Part I are: Historical; Object of the Microscopical Examination; Methods of Microscopical Examination; Microscopic Organisms in Water from Different Sources; Limnology; Geographical Distribution of Microscopic Organisms; Seasonal Distribution of Microscopic Organisms; Horizontal and Vertical Distribution of Microscopic Organisms; Odors in Water Supplies; Storage of Surface Water; Storage of Ground-Water; Growth of Organisms in Water Pipes.

Part II contains the Classification of Microscopic Organisms.

The work contains 19 plates and 21 figures and diagrams in the text. There are appendices on: Collection of Samples; Tables and Formulas; Bibliography and Glossary.

SEWERAGE.

The Designing, Construction and Maintenance of Sewerage Systems. By A. Prescott Folwell, M. Am. Soc. C. E. First Edition. Cloth, 9 x 6 ins., 372 pp. New York, John Wiley & Sons, 1898.

The author has endeavored to embody the most recent data and ideas relating to Sewerage. Sewage Disposal is only briefly considered.

The subject is subdivided as follows:

Part I, Designing: System to be Employed; Sewage Disposal; Amount of Sewage; Flow in Sewers; Flushing and Ventilation; Collecting the Data; The Design; Detail Plans; Specifications, Contract, Estimate of Cost.

Part II, Construction: Preparing for Construction; Laying Out the Work; Oversight and Measurement of Work; Practical Sewer Construction.

Part III, Maintenance: House Connections and Drainage; Sewer Maintenance.

The book contains 24 tables and is illustrated with 12 plates and 36 figures in the text. Twelve pages are devoted to an index.

EXAMINATION OF WATER (CHEMICAL AND BACTERIOLOGICAL).

By William P. Mason. First Edition. Cloth, 7 x 5 ins., 135 pp. New York, John Wiley & Sons, 1899.

Knowledge of ordinary quantitative analysis is here assumed; therefore the merest suggestions are given for determination of the mineral matters present in a water, while the items properly lying within the scope of a sanitary examination are dealt with more at length.

Of bacteriology only so much is touched upon as has been demonstrated to be of real service to the water examiner.

Contents are: Chemical Examination of Water; Bacteriological Examination of Water.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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GENERAL CRITERION FOR POSITION OF LOADS CAUSING MAXIMUM STRESS IN ANY MEMBER OF A BRIDGE TRUSS.

BY L. M. HOSKINS, ESQ.

TO BE PRESENTED APRIL 5TH, 1899.

The most dangerous load to which a railway bridge is subjected consists of a somewhat irregularly distributed series of train weights, applied to the track at the points of contact of the wheels, and to the trusses at the points of support of the track system. The actual distribution of such a load among the joints of the trusses is in some degree uncertain; some assumption must be made before the problem of computing stresses becomes determinate. In view of this uncertainty, some engineers prefer to simplify the problem by replacing the actual load by an "equivalent uniform load," or by a series of equal panel loads, or by some combination of these. Others believe that a closer approximation to actual conditions is obtained by using a series of concentrated loads representing as nearly as possible the actual weights carried by the wheels of the locomotives and cars, and assuming each load to be applied to the truss as if carried by a simple beam supported at the two adjacent panel joints.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in *Transactions*.

It is no part of the object of this paper to consider the relative merits of these different assumptions. The results of the following discussion, although especially useful in treating the case of actual wheel loads, are wholly general, so far as the character of the load series is concerned. This paper is offered as a contribution toward the unification of the theory of maximum stresses due to moving loads.

This theory, with applications to the various cases of practical importance, is quite fully treated in manuals and text-books, and in special papers dealing with particular methods.* The problem presents no serious difficulties of principle, but the treatment usually given lacks generality. This is especially true of that part of the theory which deals with the criterion for position of loads causing maximum stress, a special analysis being commonly given for each of a number of particular cases. From this lack of generality arises the chief difficulty in applying the rules deduced. The use of a special rule or formula in each of a number of cases, each involving a special notation, is likely to prove confusing, even when each case in itself is sufficiently simple.

In the endeavor to generalize the treatment of this problem, the author was led to a result simple in form and easily applicable to the various particular cases. By the use of a simple general notation, the criterion for maximum stress assumes the same form for all members, whatever their position in the truss.† The deduction of this general criterion, with examples illustrating its application in various cases, is the object of the discussion which follows.

Generality of the Discussion.

It will be well to make clear at the outset the exact degree of generality of the discussion.

As regards form of truss and choice of member, there is no restric-

* "The Stresses in Framed Structures," by A. Jay Du Bois.
"The Theory and Practice of Modern Framed Structures," by J. B. Johnson. C. W. Bryan and F. E. Turneaure.
"Text-Book on Roofs and Bridges," by Mansfield Merriman and H. S. Jacoby.
"On the Calculation of the Stresses in Bridges for the Actual Concentrated Loads," by George F. Swain. *Transactions*, Am. Soc. C. E., Vol. xvii (1887).
"A New Graphical Solution of the Problem, What Position a Train of Concentrated Loads Must Have in Order to Cause the Greatest Stress in Any Given Part of a Bridge Truss or Girder," by Henry T. Eddy. *Transactions*, Am. Soc. C. E., Vol. xxii (1890).

† The general formula expressing this criterion was given by the author in a paper presented to the Wisconsin Academy of Sciences, Arts and Letters, in December, 1893, and published in Vol. x of the *Transactions* of the Academy. The proof there given, though rigorous, was less simple than that now offered. A third proof, based upon a general graphical treatment of the problem, and depending upon elementary properties of the funicular polygon, is given in the author's "Elements of Graphic Statics," Revised Edition (1899).

tion, except that the stress must be "simply determinate." By this it is to be understood that the member under discussion may be taken as one of three, through which a section may be passed completely dividing the truss. Figs. 1 to 6 show six cases to which the analysis applies, the member under consideration being in every case marked k, and the section dividing the truss being represented by a broken line.

As regards the load series, this may consist of any distributed or concentrated loads. A concentrated load, however, is to be regarded as distributed over a finite, though short, horizontal length. This does not limit the discussion, so far as practical applicability is concerned, since the concentrated loads of practice do, in fact, act in the manner assumed. A concentrated load, in the strict mathematical sense (a finite load applied at a mathematical point), has only an ideal existence. Even such ideal loads may be regarded as limiting cases of distributed loads, and there is no difficulty in giving a discussion rigorously applicable to such an ideal load series. But the language used in the following analysis is exact only when such ideal concentrated loads are excluded.

Definitions and Notation.

Stress Moment and Moment Center.—Suppose the stress in any member to be determined by the "method of moments." For this purpose the truss is conceived to be divided by a section cutting the member in question and two others; the intersection of the axial lines of these two is taken as the origin of moments in applying the equation of equilibrium to either of the two parts of the truss. Let this origin be called the moment-center for the member under consideration, and let the moment of the required stress, with respect to this origin, be called the stress-moment. Numerically, the value of the stress-monent is equal to the sum of the moments of the external forces acting upon either of the two portions into which the truss is divided by the section; algebraically, two cases are to be distinguished by the signs plus and minus. Calling clock-wise rotation negative, let the stress-moment be called positive when it resists a tendency of the left-hand portion of the truss to rotate negatively, and of the right-hand portion to rotate positively, with reference to the moment-center.

The kind of stress in any member may be determined by inspection

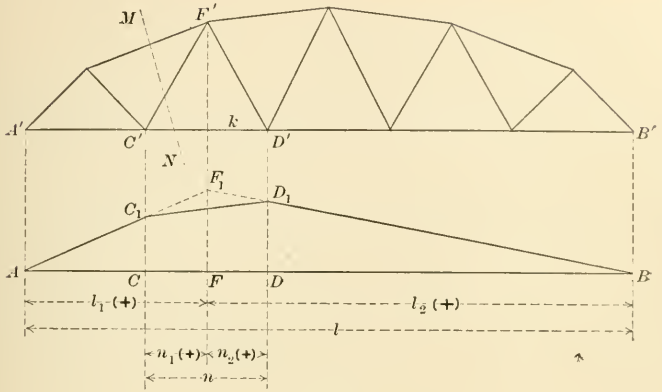

FIG. 1.

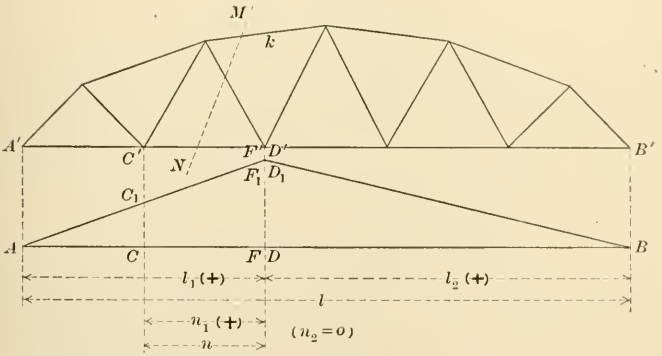

FIG. 2.

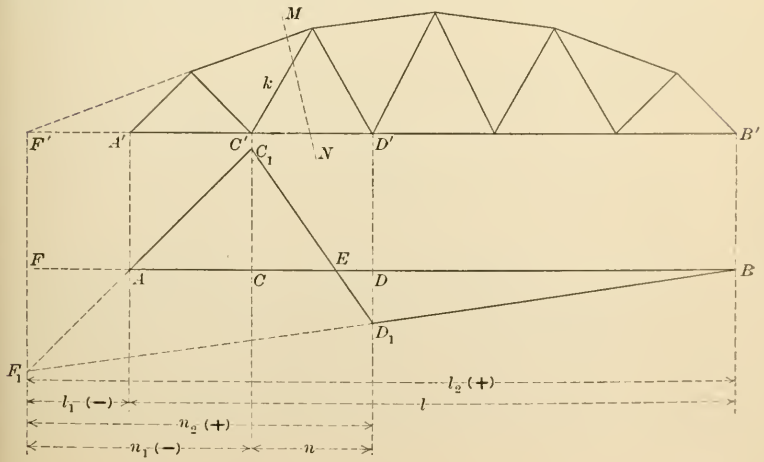

FIG. 3.

of the truss diagram, when the sign of the stress-moment and the position of the moment-center are known.

Notation.—Of the three members cut, in dividing the truss, one belongs to the chord at whose joints the moving loads are applied to the truss. Let the ends of this member be designated in all cases by C' and D', and let A' and B' be the points of support of the truss. Let the moment-center for any chosen member be designated by F'. Projecting the five points A', B', C', D' and F' upon a horizontal line, let their projections be marked A, B, C, D and F. The relative positions of these points in each of several cases are shown in Figs. 1 to 6, the member whose stress is under consideration being in every case marked k.

Let $A\,B = l$, $A\,F = l_1$, $F\,B = l_2$;
$C\,D = n$, $C\,F = n_1$, $F\,D = n_2$.

Let the signs of the four quantities l_1, l_2, n_1 and n_2, conform to the order of the letters used in defining them, the positive direction being from left toward right. Thus l_1 and l_2 are both positive if F falls between A and B; n_1 and n_2 are both positive if F falls between C and D. In all cases $l_1 + l_2 = l$ and $n_1 + n_2 = n$.

Let the load per unit length at any point be denoted by w. The only restriction as to the value of w is that it shall be either finite or zero at every point. The only case thus excluded is that of ideally concentrated loads above mentioned. At the point of application of an actual concentrated load w becomes very great, but not infinite.

Let the values of w at A, B, C and D, respectively, be denoted by w_a, w_b, w_c and w_d.

Let W = total load on the span $A\,B$;
Q = " " " panel $C\,D$;
P_1 = " " $A\,C$;
P_2 = " " $D\,B$.
Identically, $W = P_1 + P_2 + Q$.

Deduction of Criterion for Maximum or Minimum Stress.

In the following analysis, direct reference will be made to the case shown in Fig. 1, in which the four quantities l_1, l_2, n_1 and n_2 are all positive. It will be seen, however, that the reasoning applies to any of the cases shown in Figs. 2 to 6 if regard be had for algebraic signs.

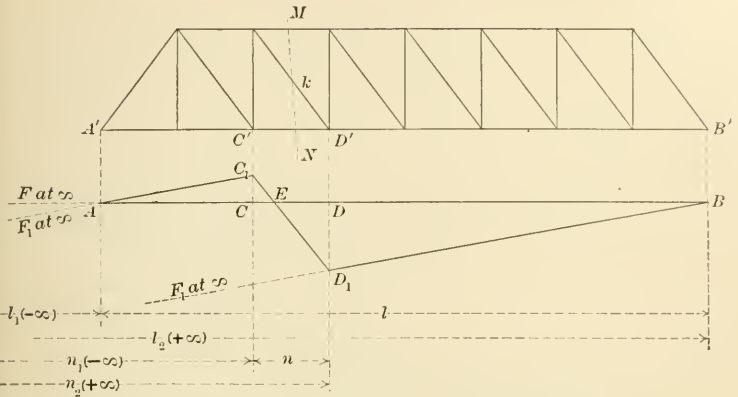

FIG. 4.

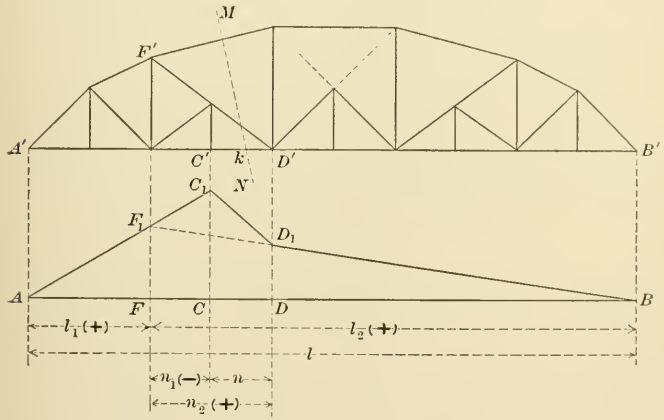

FIG. 5.

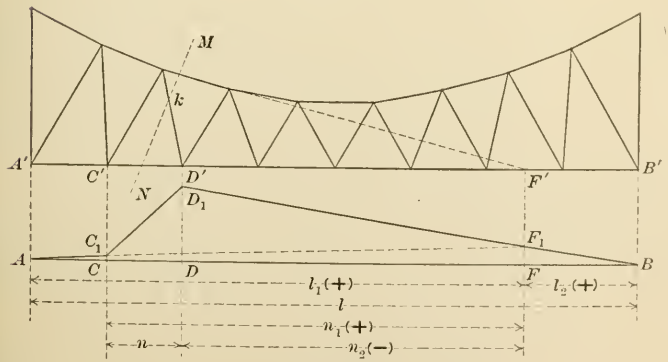

FIG. 6.

Value of Stress-Moment.—The magnitude of the resultant load on AC is P_1; let the distance of its line of action from A be p_1. Similarly, let P_2, the resultant of the loads on DB, act in a line distant p_2 from B; and let Q, the resultant of the loads on CD, act in a line distant q_1 from C and q_2 from D.

The stress-moment due to P_1 may be found by applying the principles of equilibrium to the portion of the truss at the right of the section MN (Fig. 1). The reaction at B is $\frac{P_1 p_1}{l}$; the moment of this reaction with respect to the moment-center is $\frac{l_2}{l}(P_1 p_1)$, which is therefore the value of the stress-moment due to P_1.

The stress-moment due to P_2 is found in a similar manner, by considering the equilibrium of the portion of the truss to the left of the section MN; the value being $\frac{l_1}{l}(P_2 p_2)$.

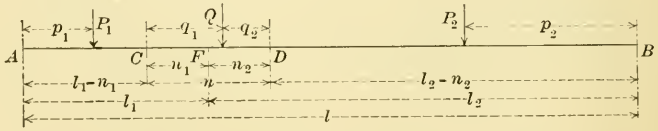

Fig. 7.

Of the load Q, the portion $\frac{Qq_2}{n}$ is applied to the truss at C, and the portion $\frac{Qq_1}{n}$ at D. The effect of the former is found as in the case of P_1; $l_1 - n_1$ taking the place of p_1 and $\frac{Qq_2}{n}$, the place of P_1; the stress-moment due to this part of Q is therefore $\frac{(l_1 - n_1)\, l_2}{nl}(Qq_2)$; while the stress-monent due to the portion at D, determined as in the case of P_2, is in like manner found to have the value $\frac{(l_2 - n_2)\, l_1}{n\, l}(Qq_1)$. These values are all positive in the case shown in Fig. 1.

The stress-moment due to the total load on the truss is the sum of these four quantities, or

$$M = \frac{l_2}{l}\left(P_1\, p_1\right) + \frac{l_1}{l}\left(P_2\, p_2\right) + \frac{(l_2 - n_2)l_1}{n\, l}\left(Q\, q_1\right) + \frac{(l_1 - n_1)l_2}{n\, l}\left(Q\, q_2\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Change of Stress-Moment Due to Small Displacement of Loads.—Let the load-series receive a small displacement δx toward the left (this being taken as the positive direction for displacements), and let δM denote the resulting increment in the value of M. To compute δM it is necessary to determine the increments of the four quantities $P_1 p_1$, $P_2 p_2$, $Q q_1$ and $Q q_2$. Of these quantities, the first is the moment of P_1 with respect to A, the second is the moment of P_2 with respect to B, the third is the moment of Q with respect to C, and the fourth is the moment of Q with respect to D.

During the displacement, loads $w_a \delta x$, $w_b \delta x$, $w_c \delta x$ and $w_d \delta x$ pass the points A, B, C and D, respectively.

After the displacement, the load which has replaced P_1 is the resultant of three components, as follows: (1) The original load P_1, acting in a line distant $p_1 - \delta x$ from A, (2) a load $w_c \delta x$ acting just at the left of C, this being the amount of load which passes C during the displacement, and (3) a negative load equal to $w_a \delta x$, acting just at the left of A, this being the amount of the load which passes off the truss at A during the displacement. The new value of $P_1 p_1$ is the sum of the moments of these three components with respect to A. Therefore, neglecting powers of δx higher than the first, we have, as the value of $P_1 p_1$, after the displacement, $P_1 (p_1 - \delta x) + w_c (l_1 - n_1) \delta x$.

Similarly, the load which has replaced P_2 is the resultant of a load P_2 acting at a distance $p_2 + \delta x$ from B, a load $w_b \delta x$ acting just at the left of B, and a negative load $w_d \delta x$ acting just at the left of D. The new value of $P_2 p_2$ is the sum of the moments of these three components about D, or $P_2 (p_2 + \delta x) - w_d (l_2 - n_2) \delta x$.

Similar reasoning gives for the new values of $Q q_1$ and $Q q_2$, $Q (q_1 - \delta x) + w_d n \delta x$ and $Q (q_2 + \delta x) - w_c n \delta x$.

The value of M after the displacement is, therefore,

$$M + \delta M = \frac{l_2}{l}\left[P_1 (p_1 - \delta x)\right] + \frac{l_1}{l}\left[P_2 (p_2 + \delta x)\right]$$
$$+ \frac{(l_2 - n_2)\, l_1}{n\, l}\left[Q (q_1 - \delta x)\right] + \frac{(l_1 - n_1)\, l_2}{n\, l}\left[Q (q_2 + \delta x)\right]$$

the terms involving w_c and w_d disappearing from the simplified result.

The increment of M caused by the displacement is

$$\delta M = \left(-\frac{l_2}{l} P_1 + \frac{l_1}{l} P_2 + \frac{l_1 n_2 - l_2 n_1}{l\, n} Q\right) \delta x.$$

Dividing by δx and passing to the limit, δx approaching zero,

$$\frac{dM}{dx} = \frac{l_1}{l}\left(P_2 + \frac{n_2}{n}Q\right) - \frac{l_2}{l}\left(P_1 + \frac{n_1}{n}Q\right) \ldots\ldots\ldots\ldots (2)$$

Condition for Maximum or Minimum Stress.—The condition for maximum or minimum value of M (and therefore also of the stress), is $\frac{dM}{dx} = 0$; which gives the equation

$$\frac{P_1 + \frac{n_1}{n}Q}{l_1} = \frac{P_2 + \frac{n_2}{n}Q}{l_2} = \frac{W}{l} \ldots\ldots\ldots\ldots (3)$$

the third member of the equation being obtained by adding the numerators and denominators, respectively, of the fractions forming the first and second members.

This general equation is satisfied whenever the stress in the member is a maximum or a minimum. That it applies to any one of the cases shown in Figs. 2, 3, 4, 5 and 6, may be verified by following the above reasoning through, taking account of the algebraic sign of every quantity dealt with.*

Special Forms of the General Criterion.—For practical use it will generally be found convenient to put equation (3) in one of the following forms:

$$P_1 + \frac{n_1}{n}Q = \frac{l_1}{l}W \ldots\ldots\ldots\ldots (3a)$$

$$P_2 + \frac{n_2}{n}Q = \frac{l_2}{l}W \ldots\ldots\ldots\ldots (3b)$$

unless the moment-center is at infinity. In this case (Fig. 4) these equations take indeterminate forms. But since when this occurs $\frac{l_1}{n_1} = 1$ and $\frac{l_2}{n_2} = 1$, both equations may be reduced to the form:†

$$Q = \frac{n}{l}W \ldots\ldots\ldots\ldots (3c)$$

In addition to the general criterion expressed by equation (3), it is necessary to have some means of distinguishing between positions giving maximum and those giving minimum values of the stress. Further, since the stress in any member will, in general, pass through a number

* For the case represented in Fig. 1, in which the member under consideration belongs to the loaded chord, l_1, l_2, n_1 and n_2 being all positive, formulas identical with equation (3) have been given by several writers. The generality of the formula seems not to have been recognized.

† Equation (3c) is identical with the condition for maximum shear in the panel containing the web member under consideration. It is, of course, only when the chords are parallel that the maximum shear and maximum stress can be assumed to occur simultaneously.

of maximum values as the loads pass over the bridge, means are needed for selecting the greatest of the possible maximum values.

A general criterion for distinguishing between positions giving maximum and those giving minimum values will be deduced below. It will be well, however, first to consider the question in a somewhat less general manner. The problem of distinguishing between maxima and minima, as well as that of selecting the greatest from the possible maximum values, may advantageously be treated by the use of "influence diagrams."

Influence Diagrams.

A very full and clear discussion of influence lines, with applications to the determination of maximum stresses in various cases of practical importance, has been given by George F. Swain, M. Am. Soc. C. E., in the paper cited on page 69. It will be advantageous to present here a brief treatment of this subject from the general point of view adopted in the foregoing analysis.

Influence Line.—The influence line, for a given member of a truss, is a line whose ordinate at any point represents the value of the stress in the member due to a unit load at that point.

General Method of Drawing Influence Line.— In the following discussion any ordinate of the influence line will be regarded as agreeing in sign with the corresponding value of the stress-moment, according to the convention already explained. Positive ordinates will be drawn upward from the reference line, negative ordinates downward.

If M denotes the stress-moment due to a unit load in a given position, and t the perpendicular distance from the moment-center to the axial line of the member under consideration, $\frac{M}{t}$ denotes the value of the ordinate of the influence line.

Referring first to the case in which l_1, l_2, n_1 and n_2 are all positive (Fig. 1), and repeating the reasoning already given in determining values of the stress-moment due to P_1, P_2 and Q, the following results are obtained:

A unit load between A and C, distant x from C, causes a stress $\frac{l_2 x}{lt}$. This changes from 0 to $\frac{l_2 (l_1 - n_1)}{lt}$ as the load passes from A to C.

Similarly, as a unit load passes from B to D, the stress due to it changes from 0 to $\frac{l_1 (l_2 - n_2)}{l t}$.

Of a unit load on $C D$, distant x from C, the portion $\frac{n - x}{n}$ comes upon the truss at C and $\frac{x}{n}$ at D. The former causes the stress $\frac{l_2 (l_1 - n_1)}{l t} \cdot \frac{n - x}{n}$, and the latter the stress $\frac{l_1 (l_2 - n_2)}{l t} \cdot \frac{x}{n}$. As the load passes from C to D, the sum of these values changes from $\frac{l_2 (l_1 - n_1)}{l t}$ to $\frac{l_1 (l_2 - n_2)}{l t}$.

From these results it is seen that the influence line consists of three straight lines $A C_1$, $C_1 D_1$, $D_1 B$; the ordinates, $C C_1$ and $D D_1$, having values:

$$C C_1 = \frac{l_2 (l_1 - n_1)}{l t}; \quad D D_1 = \frac{l_1 (l_2 - n_2)}{l t}.$$

If the various cases shown in Figs. 1 to 6 be considered, these values are found to be correct in magnitude and sign for all cases. The following general property may also readily be verified:

The lines $A C_1$ and $B D_1$ (produced if necessary) have a common ordinate at the point F, its value being $\frac{l_1 l_2}{l t}$.

This property gives the following general rule for drawing the influence line:

At F erect an ordinate equal in magnitude and sign to $\frac{l_1 l_2}{l t}$, and draw straight lines from its extremity to the points A and B. Connecting C_1 and D_1, the points in which these lines intersect ordinates erected at C and D, respectively, the line $A C_1 D_1 B$ is the influence line.

It will be seen that, so long as the moment-center falls within the span, so that both l_1 and l_2 are positive, the ordinates of the influence line are everywhere positive. This is the case in Figs. 1, 2, 5 and 6. There are, however, two types of influence line with ordinates everywhere positive, corresponding to the cases in which F falls within the panel CD (Fig. 1), and without the panel (Figs. 5 and 6). The limiting case between these two is shown in Fig. 2.

If the moment-center falls without the span, making l_1 or l_2 negative, the ordinates of the influence line are positive in one portion of

the span and negative in the remaining portion. Fig. 3 represents the case in which l_1 is negative, the ordinate CC_1 being positive and DD_1 negative. The line $C_1 D_1$ crosses $A B$ at a point E, between C and D.

If the point F falls at infinity, as in Fig. 4, the above general rule for drawing the influence line by means of the ordinate at F becomes inapplicable. The lines $F_1 A$, $F_1 B$ are in this case parallel, and may be drawn by means of the general values above given for the ordinates at C and D. It may be noticed also that if $A C_1$ be prolonged, its ordinate at B has the value $\frac{l_2}{t}$; and if $B D_1$ be prolonged, its ordinate at A has the value $\frac{l_1'}{t}$. These values are determinate even when l_1, l_2 and t are infinite.

When the load series consists of concentrated loads, the question whether any certain position in which the general equation (3) is satisfied gives a maximum or a minimum value of the stress may usually be determined by simple inspection of the influence diagram. It is also often possible to determine, with high probability, which of several maximum values is greatest, without actually determining the values. These questions are now to be considered.

Discrimination Between Maximum and Minimum Values.

Application of Influence Diagram.—Referring to Fig. 1, and observing the slope of each of the three straight lines forming the influence diagram, consider how the stress in the member k, due to a single load, varies as the load moves uniformly from B toward the left. During the motion from B to D the stress increases at a uniform rate; from D to C it decreases at a uniform rate; from C to A it decreases uniformly at a greater rate. If the load moves from A to B, these effects are reversed.

Next consider the effect of displacing a series of loads. As above, let the total loads on $A C$, $C D$ and $D B$, respectively, be denoted by P_1, Q and P_2. If the loads are displaced toward the left, the stress due to P_2 increases, while the stress due to P_1, and also that due to Q, decreases. The resultant of these three effects may be either an increase or a decrease; but the resultant rate of change of the stress (whether increase or decrease) remains constant as long as P_1, P_2 and Q remain constant, that is, as long as no load passes any one of the four points A, C, D and B.

As the loads approach a position causing a maximum stress, the value of the stress increases; as they recede from such a position, the value of the stress decreases. The reverse changes take place as the loads approach and recede from a position of minimum stress.

First, suppose the stress is increasing as the loads move toward the left. If a load comes on the truss at B, the rate of increase of the stress will become greater. The same will be true if a load passes off at A. If a load passes C or D, the rate of increase of the stress becomes less, and this effect may be sufficiently great to change the increase into a decrease; if so, the stress passes through a maximum value. Hence, only when a load is at C or at D can the stress be a maximum.

Next, suppose the stress is decreasing as the loads move toward the left. If a load passes C or D, the rate of decrease becomes greater. If a load comes on the truss at B, or if one leaves the truss at A, the rate of decrease becomes less, and this effect may be so great as to change the decrease into an increase; if so, the stress passes through a minimum value. Therefore, only when a load is at A or at B can the stress be a minimum.

Like reasoning obviously applies to the cases represented in Figs. 2, 3, 4, 5 and 6.*

Thus, in Fig. 2, a maximum can occur only with a load at D. A load at C can give neither a maximum nor a minimum, since $A\,C_1$ and $C_1\,D_1$ form a continuous straight line.

In the case of a web member, represented in Fig. 3, the stress may be either tension or compression. The kind of stress corresponding to a positive stress-moment (tension in this case), can have a maximum value with a load at C or at B, a minimum value with a load at A or at D. For the opposite kind of stress, these statements must be reversed. Like statements hold for the case in which F is at infinity (Fig. 4).

In Fig. 5, a maximum value of the stress in the member k can occur only when a load is at C; a minimum value may occur when a load is at A, at D or at B.

In Fig. 6, a maximum value can occur with a load at D, a minimum value with a load at A, at C or at B. This form of truss, though not of practical importance, is shown in order to illustrate the generality of the method. Its similarity to the case shown in Fig. 5 is apparent from

*The actual value of the rate of increase of the stress as the loads move may be computed from the slope of the three portions of the influence line. Using the values of the ordinates at C and D, as above given, this method leads easily to equation (2), and thus also to the general criterion (3).

a comparison of the influence diagrams. It may be remarked that, as a special case of Fig. 6, the point F may coincide with B (or with A), making l_2 (or l_1) zero. The ordinate $F F_1$ would be zero, and the influence line would have to be constructed from the values of the ordinates at C and D, one of which would be zero.

Critical Points.—It appears from the foregoing discussion that, when the load-series consists wholly of concentrated loads, the stress can have a maximum or a minimum value only when a load is passing some one of the four points A, B, C and D. This might be inferred also from equation (3), since only in exceptional cases can this equation be satisfied without there being a load at one of these points. Such a load may, by a slight displacement, be divided in any desired ratio between the adjacent segments of the span.

These four points may be called critical points, and may be distinguished as "maximum points" and "minimum points," according as the passage of a load through them is associated with a maximum or a minimum value of the stress.

The foregoing discussion shows that, at a maximum point, the influence line is convex upward (*i. e.*, toward the positive side), while at a minimum point it is concave upward. In applying this statement to the points A and B, the influence line is to be extended beyond the span as a horizontal line.

When only a single one of the four critical points is occupied by a load, or if such occupied points are either all maximum points or all minimum points, the form of the influence line thus shows at a glance whether the stress is a maximum or a minimum. To make the discussion complete it is necessary to consider cases in which equation (3) is satisfied while loads are passing both maximum and minimum points.

General Criterion.—In order to deduce a completely general rule for discriminating between maximum and minimum values, consider equation (2) deduced above:

$$\frac{d\,M}{d\,x} = \frac{l_1}{l}\left(P_2 + \frac{n_2}{n}\,Q\right) - \frac{l_2}{l}\left(P_1 + \frac{n_1}{n}\,Q\right) \ldots\ldots\ldots\ldots (2)$$

This value of $\frac{d\,M}{d\,x}$ remains constant except when the load (either concentrated or distributed), passes some one or more of the "critical" points A, B, C or D, thus changing the values of one or more of the quantities P_1, P_2 or Q.

Assuming a small displacement of the load-series, let loads W_a, W_b, W_c and W_d, respectively, pass the four critical points A, B, C and D. The change in the value of $\frac{dM}{dx}$ may be found from the changes in P_1, P_2 and Q. For a positive displacement (the loads moving toward the left),

P_1 changes by the amount $W_c - W_a$;
P_2 " " $W_b - W_d$;
Q " " $W_d - W_c$.

Hence the change (written as an increase) in the value of $\frac{dM}{dx}$ is

$$\delta\left(\frac{dM}{dx}\right) = \frac{l_1}{l}\left[W_b - W_d + \frac{n_2}{n}(W_d - W_c)\right] - \frac{l_2}{l}\left[W_c - W_a + \frac{n_1}{n}(W_d - W_c)\right];$$

which reduces to

$$\delta\left(\frac{dM}{dx}\right) = \frac{l_2}{l}W_a + \frac{l_1}{l}W_b - \frac{n_2}{n}W_c - \frac{n_1}{n}W_d \ldots\ldots (4)$$

According as this value is positive or negative, the value of $\frac{dM}{dx}$ increases or decreases as the loads pass through the supposed position.

As a position of maximum stress is passed, $\frac{dM}{dx}$ decreases; as a position of minimum stress is passed, $\frac{dM}{dx}$ increases.

This is the general criterion for discriminating between maximum and minimum values of the stress. The application to special cases, in which l_1, l_2, n_1 and n_2 are known in magnitude and sign, presents no difficulty.

The conclusions above reached from the study of the influence diagrams for the six cases, may easily be verified by inspection of equation (4). Thus, if l_1, l_2, n_1 and n_2 are all positive (Fig. 1), the first and second terms in the value of $\delta\left(\frac{dM}{dx}\right)$ are intrinsically positive, while the third and fourth are negative. Therefore, for a maximum stress either W_c or W_d must be present, while for a minimum stress either W_a or W_b must be present. In the case shown in Fig. 2, the third term in the value of $\delta\left(\frac{dM}{dx}\right)$ drops out, n_2 being zero; the conclusions are otherwise the same as in the preceding case.

In Fig. 3, l_1 and n_1 being negative, the "A" and "D" terms in the value of $\delta\left(\frac{dM}{dx}\right)$ are intrinsically positive, while the "B" and "C"

terms are negative. Hence A and D are minimum points, while B and C are maximum points. The same conclusions apply to the case shown in Fig. 4, although every term in the second member of equation (4) is infinite.

In Fig. 5, n_1 is negative, while l_1, l_2 and n_2 are positive. Therefore in equation (4) the "A," "B" and "D" terms are positive, while the "C" term is negative. Hence C is the only maximum point.

In Fig. 6, n_2 being negative, the only negative term in equation (4) is the "D" term. Hence D is the only maximum point.

If all but one of the four terms in the value of $\delta\left(\frac{d\,M}{d\,x}\right)$ are zero when the condition for maximum or minimum stress (equation 3) is satisfied, the discrimination between maximum and minimum values is thus very simple, being readily accomplished, either by simple inspection of the influence diagram, or by noticing the sign of the single term in the second member of equation (4). If two or more terms remain, equation (4) must be applied.

Distributed Loads.—Consider next the case in which there is a distributed load at each of the four points A, B, C and D. Equation (4) is still applicable, but its form may conveniently be changed. In accordance with the notation used in the deduction of equation (1), let w denote the load per unit length at any point, and let w_a, w_b, w_c and w_d be the values of w at A, B, C and D. If the loads receive a positive displacement $\delta\,x$, we must put in equation (4):

$$W_a = w_a\,\delta\,x,\ W_b = w_b\,\delta\,x,\ W_c = w_c\,\delta\,x \text{ and } W_d = w_d\,\delta\,x.$$

The result is:

$$\delta\left(\frac{d\,M}{d\,x}\right) = \left(\frac{l_2}{l}\,w_a + \frac{l_1}{l}\,w_b - \frac{n_2}{n}\,w_c - \frac{n_1}{n}\,w_d\right)\delta\,x \quad \ldots\ldots (4a)$$

or

$$\frac{d^2\,M}{d\,x^2} = \frac{l_2}{l}\,w_a + \frac{l_1}{l}\,w_b - \frac{n_2}{n}\,w_c - \frac{n_1}{n}\,w_d \quad \ldots\ldots\ldots (5)$$

A position of the loads satisfying equation (3) gives a maximum or a minimum value of the stress, according as the value of $\frac{d^2\,M}{d\,x^2}$ is negative or positive. If the value of w is known at every point, the application of equation (5), to any particular case in which l_1, l_2, n_1 and n_2 are known, presents no difficulty.

Combination of Concentrated and Distributed Loads.—Another case is that in which there are concentrated loads at certain of the critical

points, while at others the load is distributed. This occurs when the load consists of one or more locomotives followed by a uniformly distributed train-load, as often prescribed in bridge specifications. The value of $\delta \left(\frac{d M}{d x}\right)$ will then contain part of the terms in equation (4) and part of those in equation (4*a*). Since the latter terms contain the factor δx, they will be very small in comparison with the former, and may, therefore, generally be disregarded in applying the equation.

Determination of Greatest Maximum.

In those cases in which loads in all positions cause the same kind of stress in the member under consideration, it is evident that for greatest stress the span should be loaded as fully as possible, and that the heaviest loads should be so placed as to produce as great an effect as possible. Thus in Figs. 1, 2 and 6 the heavy loads should be near D; in Fig. 5, near C. This is, of course, only a rough, general rule.

When a reversal of stress is possible, as in Figs. 3 and 4, the two kinds of stress must be considered separately. Thus, for one kind of stress, loads should cover only the portion $A E$; for the other kind, only the portion $E B$. These rules are not absolute; it may be that, if loads are carried beyond E, the diminution of stress due to such loads will be more than counterbalanced by an increase due to bringing other loads nearer to their positions of greatest effect.

By inspection of the influence diagram it is thus possible to determine, with great probability in many cases, which of two maximum values is greater, without actually computing the values. In some cases, however, it may be necessary to compute the stress for each of two or more positions, in order to determine with certainty which maximum value is the greatest.

General Results.

The determination of the position of a series of moving loads which will cause the greatest stress in any member of a truss thus involves the following steps:

(1) The approximate determination of the required position by inspection of the influence diagram.

(2) The testing of the exact criterion expressed by equation (3), or one of the equivalent equations (3*a*), (3*b*), (3*c*).

(3) The application of equation (4) or equation (5) to determine with certainty whether the stress is a maximum or a minimum. In the majority of practical cases this is, however, unnecessary, the inspection of the influence diagram being sufficient.

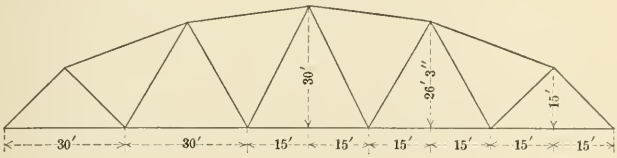

Fig. 8.

Numerical Applications.

In order to illustrate clearly the application of the general criterion to different cases, numerical examples will now be solved, involving the cases shown in Figs. 1, 3, 4 and 5.

The load-series will be assumed to consist of two locomotives followed by a uniformly distributed load, the loads and intervals being given in Fig. 9. In practice, a diagram should be drawn, showing the lines of action properly spaced, and a skeleton diagram of the truss

10	35	60	85	110		125	140	155	170	180		205	230	255	280		295	310	325	340	*Total Load (thou's of lbs.)*
10	25	25	25	25		15	15	15	15	10		25	25	25	25		15	15	15	15	*Loads (thou's of lbs.)*
1	2	3	4	5		6	7	8	9	10		11	12	13	14		15	16	17	18	
○	◯	◯	◯	◯		○	○	○	○	○		◯	◯	◯	◯		○	○	○	○	*2.5 per foot*
7.5′	5′	5′	5′	9′		5′	5′	5′	8′	7.5′		5′	5′	5′	9′		5′	6′	5′	5′	
7.5′	12.5′	17.5′	22.5′			31.5′	36.5′	42.5′	47.5′	55.5′		63.0′	68.0′	73.0′	78.0′		87.9′	92.0′	98.0′	103.0′	*Dist. from 1st. Load*

Fig. 9.

prepared on a movable strip of paper or on tracing cloth, to facilitate the trial of different positions.

(1) *Member of Loaded Chord.*—Consider, first, the case shown in Fig. 1, the dimensions of the truss being as marked on Fig. 8. For greatest stress in the member k the truss must be fully loaded, and the effect of the heaviest loads will be greatest if these are near the point D. A maximum value can occur only with a load at C or at D. It remains to test the general equation (3) for such positions as these general considerations suggest.

From the dimensions of the truss we have $l = 150$ ft., $l_1 = 45$ ft.,

$l_2 = 105$ ft., $n = 30$ ft., $n_1 = 15$ ft. and $n_2 = 15$ ft. Equation (3a) therefore becomes $P_1 + \frac{1}{2} Q = \frac{3}{10} W$.

It is seen that there is a conflict between the requirements that the span shall be fully loaded, and that the heaviest loads shall be near D.

It is found that there are two positions in which the conditions for maximum stress are satisfied—with either the fourth or the fifth load at C. Between two maxima there must be a minimum; and it is found that between the two positions of maximum stress there is another position in which equation (3a) is satisfied. The numerical data for testing these three positions and two others are shown in Table No. 1.

Table No. 1.

Position.	W	$\frac{l_1}{l} W$	P_1	Q	$P_1 + \frac{n_1}{n} Q$	
Third load at C..	401 250	120 375	35 000 60 000	105 000 120 000 80 000 95 000	87 500 95 000 100 000 107 500	Criterion not satisfied.
Fourth load at C	413 750	124 125	60 000 85 000	95 000 110 000 70 000 85 000	107 500 115 000 120 000 127 500	Criterion satisfied. Maximum.
Fourth load 4.5 ft. left of C....	425 000	127 500	85 000	85 000	127 000	Criterion satisfied. Minimum.
Fifth load at C...	426 250	127 875	85 000 110 000	85 000 60 000	127 500 140 000	Criterion satisfied. Maximum.
Tenth load at D..	433 750	130 125	110 000	60 000 70 000	140 000 145 000	Criterion not satisfied.

It happens that when the fourth load is at C, the ninth is at D. Hence, Q may include any arbitrary portion of each of these two loads. Thus, if the whole of the fourth load is included in Q, the value of P_1 is 60 000 lbs., while Q may be either 95 000 lbs. or 110 000 lbs. or anything between, depending upon what portion of the ninth load is included in Q. If the whole of the fourth load is included in P_1, the value of P_1 is 85 000 lbs., while Q has any value from 70 000 to 85 000 lbs. The values corresponding to these four limiting cases are shown in Table No. 1. Evidently $P_1 + \frac{n_1}{n} Q$ may have any value between 107 500 lbs. and 127 500 lbs.; and since $\frac{l_1}{l} W = 124\,125$ lbs., the equation $P_1 + \frac{n_1}{n} Q = \frac{l_1}{l} W$ is satisfied.

Since, in the position of minimum stress the point C is 4.5 ft. from the fourth load and only 0.5 ft. from the fifth, it may be inferred that the stress is greater when the fourth load is at C than when the fifth is there. This may be verified by computing the values of the stress.

The general criterion is satisfied in no other position which seems favorable for greatest stress. The conclusion is that the member has its greatest stress when the fourth load is at C.

(2) *Web Member.*—Consider next the case of a web member, shown in Fig. 3, the dimensions of the truss being as marked in Fig. 8. The influence line shows that the stress is subject to reversal, so that tension and compression must be considered separately. It will suffice to consider the case of compression, corresponding to negative values of the stress-moment.

Inspection of the influence line shows that the load should cover the portion EB of the span; that a load has its greatest effect when at D; and that a maximum compression can occur only when a load is at D (the case of a maximum with a load at A being excluded because the portion of the span AE should be free from loads). It remains to apply the exact criterion.

A trial of successive positions shows that the criterion for maximum stress is satisfied when the second load is at D, but not in any other position which seems favorable for greatest stress. The conclusion is that this position gives the greatest compression in the member. The data for testing three positions are shown in Table No. 2.

From the dimensions of the truss as shown in Fig. 8, we have $l =$ 150 ft., $l_1 = -25$ ft., $l_2 = 175$ ft., $n = 30$ ft., $n_1 = -55$ ft., $n_2 = 85$ ft., $\frac{l_1}{l} = -\frac{1}{6}$ and $\frac{n_1}{n} = -\frac{11}{6}$.

TABLE No. 2.

Position.	W	$\frac{l_1}{l} W$	P_1	Q	$P_1 + \frac{n_1}{n} Q$	
First load at D..	295 000	—49 167	0	0 10 000	0 — 18 333	Criterion not satisfied.
Second load at D.	310 000	—51 667	0	10 000 35 000	— 18 333 — 64 167	Criterion satisfied. Maximum.
Third load at D..	325 000	—54 167	0	35 000 60 000	— 64 167 —110 000	Criterion not satisfied.

(3) *Web Member in a Truss with Parallel Chords.*—In Fig. 4, a negative stress-moment corresponds to tension in the member k. For greatest tension in this member, the influence line shows that only the portion $E\,B$ of the span should be loaded, and that a load must be at D.

Let the span consist of eight panels of 30 ft. each. Then $l = 240$ ft., $l_1 = -\alpha$, $l_2 = +\alpha$, $n = 30$ ft., $n_1 = -\alpha$, $n_2 = +\alpha$, $\frac{l_1}{n_1} = 1$, and $\frac{l_2}{n_2} = 1$. The criterion for maximum or minimum stress reduces to equation (3c), that is,

$$Q = \frac{n}{l}\ W = \frac{1}{8}\ W.$$

It is found that this equation is satisfied when either the third or the fourth load is at D. Both these positions give maximum values of the stress. Between them is a position of minimum stress, which, by applying equation (3 c), is found to be that in which the third load is 1.5 ft. to the left of D. Comparing this with the two positions of maximum stress, it seems highly probable that the greater of the two maximum values is that which occurs when the fourth load is at D. The numerical data for these three positions are shown in Table No. 3.

TABLE No. 3.

Position.	W	$\frac{n}{l}\ W$	Q	
Third load at D.....	476 250	59 530	35 000 60 000	Criterion satisfied. Maximum.
Third load 1.5 ft. left of D..	480 000	60 000	60 000	Criterion satisfied. Minimum.
Fourth load at D.....	488 750	61 094	60 000 85 000	Criterion satisfied. Maximum.

(4) *Chord Member, Truss with Subordinate Bracing.*—The case represented in Fig. 5 is peculiar in that l_1 and n_1 have opposite signs. The form of the influence diagram shows that for greatest stress the span should be fully loaded, and that a load must be at C for a maximum stress.

Let the span consist of five main panels of 60 ft. each, subdivided into ten panels of 30 ft. each; then $l = 300$ ft., $l_1 = 60$ ft., $l_2 = 240$ ft., $n = 30$ ft., $n_1 = -30$ ft. and $n_2 = 60$ ft. Equation (3*a*) reduces to

$$P_1 - Q = \frac{1}{5}\ W.$$

Trying successively the positions which appear favorable for greatest stress, it is found that the criterion is satisfied when the thirteenth load is at C, but not with either the twelfth or the fourteenth at C. This is shown in Table No. 5.

TABLE No. 5.

Position.	W	$\frac{1}{5}W$	P_1	Q	$P_1 - Q$	
Twelfth load at C..	765 000	153 000	205 000	120 000	85 000	Criterion not satisfied.
			230 000	105 000	100 000	
				95 000	135 000	
				80 000	150 000	
Thirteenth load at C.....	777 500	155 500	230 000	110 000	120 000	Criterion satisfied.
			255 000	95 000	135 000	Maximum.
				85 000	170 000	
				70 000	185 000	
Fourteenth load at C.....	790 000	158 000	255 000	85 000	170 000	Criterion not satisfied.
			280 000	60 000	220 000	

When the thirteenth load is at C, there is also a load at D. Since D is a "minimum-point," it is strictly necessary to apply equation (4) to determine with certainty whether the stress is a maximum or a minimum. We have $W_c = 25\,000$ lbs., $W_d = 15\,000$ lbs., $\frac{n_2}{n} = 2$, and $\frac{n_1}{n} = -1$; and equation (4) becomes

$$\delta\left(\frac{d\,M}{d\,x}\right) = -\,50\,000 + 15\,000 = -\,35\,000.$$

Since this value is negative, while M is positive, the stress is a maximum.

The conclusion is that the greatest stress occurs when the thirteenth load is at C.

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PAPERS AND DISCUSSIONS.

Note.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

DRY DOCKS—STONE vs. WOOD.

An Informal Discussion.*

By Messrs. W. L. Cathcart, George W. Melville, M. T. Endicott, R. G. Packard, P. C. Asserson, J. G. Tait, John Kennedy, James Ritchie, Francis Collingwood, Charles H. Haswell, Francis T. Bowles and Alfred P. Boller.

Presented at a Special Meeting of the Society on February 10th, 1899.

Mr. Cathcart. William Ledyard Cathcart, Esq.—The modern war vessel is practically helpless without the dry dock, in which she must be placed periodically for the cleansing of the hull, and, with more or less frequency, for the examination and repair of all exterior parts below the water line. The expansion of the fleet and that of the system of national dry docks should, then, go hand in hand; and yet, with a rehabilitated navy, which is already formidable in strength, the United States is far behind the other great powers of the world, not only in the number, but apparently in the material, of its dry docks. Of the eleven which this Government now controls and operates, only four are of masonry, the remainder being timber constructions; and it is proposed further, in recent legislation, to build three more docks of wood.

Practice of Foreign Governments.—The practice of foreign governments differs widely from this. In Europe, not only national graving

* The discussion of this subject, for which no formal paper was presented, is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, p. 71.)

Communications on this subject received prior to April 22d, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

docks, but all great modern works of this kind are constructed of stone or concrete masonry. Thus, the British Government owns or controls sixty-two graving docks—some of which are locks, which answer the purposes of dry docks—and only one of these is of timber, that at St. John's, Newfoundland, which was built by the Colonial Government. France has thirty-six, Germany has eleven, and Italy has twelve, graving docks, all of which, in the three countries, are of masonry. In Germany, a large dry dock was recently designed for a corporation there, which was to be subsidized by the Government. When, however, the dock company proposed to use timber as the material, the Government declined to aid, and it was only upon condition that the dock be of masonry that the subsidy was finally granted. Mr. Cathcart.

It will be seen, then, that the United States Government is the only one in the world which has constructed, directly, a timber dry dock. The governments of Europe have considered it the policy of wisdom to build these important works of the most enduring materials, without regard to cost. This decision has been reached, doubtless, in view, not only of the natural decay, the frequent repairs and the large expense thereof, to timber dry docks, but, as well, because of the superior stability, permanence and safety of a dock which is built of stone or concrete.

Timber vs. *Stone in Dry Dock Construction, with Regard to Deterioration.*—Timber dry docks are subject to decay, especially in the upper portions of the works, which are alternately wet and dry. The deterioration is especially swift in a warm, moist climate; but, even in northern latitudes, where decay is much slower, docks built of wood require, in about 25 years, very extensive renewals. In southern waters, submerged timber, also, perishes rapidly from the attacks of the teredo.

On the other hand, the masonry dock, if built of granite or other enduring stone and mortar of the best Portland cement, is a permanent work. In the older stone dry docks of the United States, where the mortar was made with native hydraulic cements, the masonry has suffered some injury to the joints of the entrance works, from which there was resultant leakage. This has occurred to a small extent, with the docks at Boston and Norfolk; but there has been no serious dislocation of the masonry, and the small amount of leakage has injured the structure in no way and has involved little expense for renewal. These two docks, although sixty-five years old, have had inconsiderable repairs to the masonry. The same action, from the same cause, took place with the stone dock at the Brooklyn Navy Yard. In this case, owing to the strong pressure from springs upon the site, the disintegration of joints by frost was considerable, and there was heavy leakage. Repairs, made in the best manner, were, however,

Mr. Cathcart

wholly effectual. As a general principle, it may be broadly stated that it is possible to make a masonry dry dock absolutely water tight.

Cost of Maintenance of the Various Stone and Timber Dry Docks of the United States Navy.—The relative values of stone and timber in dry-dock construction may be considered, first, with regard to the cost of maintenance of the docks, and, secondly, with regard to their stability and safety. The first consideration only will be discussed herein.

As to this maintenance, it is obvious that the proper basis of comparison is the relative costs of repairs to the structures proper, not including the accessories, such as pumping plant, caisson, floating gates, etc., inasmuch as the cost of maintenance of the latter should be the same in all similar docks, without regard to the material of construction of the dock proper. The history of the national dry docks of the United States is instructive in this respect.

Stone Dry Dock at the Navy Yard, Boston.—This dock was completed in March, 1834. The record of its repair and maintenance prior to 1866 is incomplete and not readily accessible. From that time, however, to September 1st, 1897, a period of 31 years, the expenditures for these items were as follows:

Repair and maintenance of dock proper		$62 000
Pumping plant	$53 764	
Caisson and swinging gate	14 899	
		68 663
Total		$130 663
Average, for dry dock proper, per year		2 000

It will be observed that less than half of the total amount, or 47%, was expended upon the dry dock proper.

Stone Dry Dock at Norfolk, Va.—This dock was built of cut granite, upon the same plans as the dock at Boston. The early records as to its maintenance are not available. From 1882 to 1897, a period of 15 years, the expenditures were as follows:

Repair and maintenance of dock proper		$18 776
Pumping plant	$12 108	
Caissons or floating gates	30 546	
		42 654
Total		$61 430
Average, for dry dock proper, per year		1 251

It will be noted that the expenditures upon the dock proper are but 30% of the total.

Stone Dry Dock at Brooklyn, N. Y.—This dock was completed in 1851. The figures for this dock do not compare favorably with those already given, owing to the character of the soil in which it was con-

structed and to the quality of the cement used in the joints. Many difficulties were encountered with the foundations, arising from the large springs upon the site, which were never driven entirely from the locality. The stratum through which the springs flowed was 30 ft. below the foundations. As the water was entirely fresh, as its volume was not affected by the rise and fall of the tides, and as the temperature differed from that of the water in the bay, it is unquestionable that the flow came from a source much higher than tide-water. Mr. Cathcart.

Again, the mortar used in the construction of this dock was not made from the fine Portland cement manufactured at the present day, but from the hydraulic cements which were used in all structures of its type at the time of its building.

The heavy pressure from the springs referred to, combined with the inferior quality of the cement, and the action of frost, tended to produce disintegration of the masonry, to some extent, with, in consequence, very considerable leakage, thus necessitating during the fiscal years 1886–87 and 1887–88, extensive repairs, amounting to about $88 000. It became essential to take up and relay most of the masonry at the entrance and a small portion at the head. The repairs, made with great care, and with the best Portland cements, were wholly successful, and, during the 10 years which have passed since their completion, there have been no signs of failure of the masonry, to any extent.

Despite these adverse conditions, the total cost of repairs and maintenance of the dock proper, for 45 years ending with 1897, has been but $210 618, or an average of $4 680 per year. The cost of repairs to the caisson, gates, etc., during the same period, was $69 762. The cost of maintenance of the pumping plant during this time is not available.

Stone Dry Dock at Mare Island, Cal.—This dock was completed in 1891. Since that time, up to, and including 1897, the expenditures have been as follows:

Repair and maintenance of dock proper		$1 613
Pumping plant	$8 258	
Caisson or floating gate	4 534	
		12 792
Total		$14 405
Average, for dry dock proper, per year, during this period of 7 years		$230

The expenditures upon the dock proper were, in this case, but 11% of the total. It will be observed that the figures given do not include the cost of a 40-ton crane and track, and of a spare caisson which was

Mr. Cathcart. provided as a relief in case of repairs to the original. In some published comparisons, to be later referred to, a large proportion of these items would seem to have been charged erroneously against repairs to the dry dock proper.

Similar expenditures upon the wooden dry docks of the Navy will now be considered.

Timber Dry Dock No. 2, Navy Yard, N. Y.—This dock was completed in 1890. The expenditures upon it for the first 7 years of its life were as follows:

Repair and maintenance of dock proper		$1 982
Pumping plant	$3 086	
Caisson, gates, etc	12 706	
		15 792
Total		$17 774

This would give, during this period, an average cost of repairs per year, to the dock proper, of but $283; but the story does not end here. The timber work of this dock has deteriorated considerably, and an inward movement of the abutments has taken place. The dock, in consequence of these defects, was surveyed by a board of civil engineers in 1898, and its condition was such that they recommended an entire remodeling and reconstruction of the entrance in concrete instead of timber. At its last session, Congress appropriated the sum of $300 000 for making these repairs.

If this work be completed in the year 1900, the dock will have cost, for repairs and maintenance, during the first 10 years of its life, $317 774; and, for the dry dock structure proper, $301 982; or an average of $30 198 per year.

It is quite true that this large sum for repairs in concrete is much more than would be the cost of the work in timber; and yet the Navy Department has considered the expenditure necessary in order to make the dock wholly trustworthy.

Timber Dry Dock at the Navy Yard, League Island.—This dock was completed in 1891. During its first 6 years of life, the expenditures upon it were as follows:

Repair and maintenance of dock proper		$10 202
Pumping plant	$3 632	
Caisson	113	
		3 745
Total		$13 947
Average, for dry dock proper, per year		$1 700

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The upper portion of the timber work, however, has decayed, to a very great extent, and there has been an inward movement of the abutments which has threatened an increase sufficient to prevent the seating of the caisson in its groove. In view of these facts, Congress, at its last session, appropriated $64 000 for repairs. If these repairs be completed during the present year, the dock will have cost, within the first 9 years of its life, $77 947; and the repairs and maintenance of the structure proper, $74 202, an average of $8 245 per year.

Timber Dry Dock No. 3, Navy Yard, N. Y.—This dock was completed in the spring of 1897. In the 2 years of its existence, it has cost $171 000 for repairs, an average of $85 500 per year.

These extraordinary expenditures, it is but just to say, resulted from a failure of the dock, due to faults in the construction, not in the design. If the work had been faithfully carried out, the injuries would not have occurred, and the expenditures for repairs, during these 2 years, would probably have been merely nominal. On the other hand, the failure was of a character which could not have occurred in a masonry or concrete dock.

Timber Dry Dock, Navy Yard, Norfolk, Va.—This dock was completed in 1889. The expenditures upon it to June, 1897, or during the first 8 years of its existence, were as follows:

Repair and maintenance of dock proper		$13 576
Pumping plant	$8 312	
Caisson or floating gate	1 532	
		9 844
Total		$23 420
Average, for dry dock proper, per year		$1 697

Since the date given above, however, Congress has made two appropriations for repairs to the structure, aggregating $35 000. These will have been expended within a few months, making the total cost for repairs to the dry dock structure proper during its 10 years of life $48 576, an average of $4 857 per year.

Timber Dry Dock at Port Royal, S. C.—This dock was completed in 1895. For the first 2 years the repairs upon the dry dock structure proper amounted to $4 262, an average of $2 131 per year.

Since this period, however, about $12 000 have been expended in repairs to the structure, making for 4 years a total of $16 262, or an average of $4 065 per year.

In summing up, finally, the relative cost of maintenance of the stone and wooden docks of the navy, the stone docks at Boston, Norfolk and Mare Island will be considered and that at New York omitted, because, from the character of its foundations and the cement used originally in its construction, it is not fairly representative.

Mr. Cathcart. From the wooden docks there will be omitted also Timber Dry Dock No. 3, at New York, the large expenditures for which are due to faults in building; but the timber docks at New York, League Island, Norfolk and Port Royal will be included (see Table No. 1).

It will be seen that the dates previously set forth herein, cover for the three stone docks, an average period of $17\frac{2}{3}$ years, and for the four timber docks an average period of but $8\frac{1}{4}$ years, which period is, however, as long as the age of the latter permits. The total expenditure on the structures proper has been, for the three stone docks, $82 389, or an average of $27 463 per dock during the average period of $17\frac{2}{3}$ years. The corresponding cost of the four timber docks has been $441 022, or an average of $110 255 per dock during their average life of $8\frac{1}{4}$ years. The average per dock structure proper per year on the basis of the figures given above is, for the stone docks, $1 558; for those of timber, $13 364, or more than eight and one-half times as much. Results such as these would seem to make most questionable the further use of timber in the building of national docks.

Disregarding, however, all comparisons, the fact remains that the four timber docks referred to have cost in repairs to the structures proper during an average life of $8\frac{1}{4}$ years in expenditures and sums appropriated the large amount of $441 022. As has been said, the failure of Timber Dry Dock No. 3, at New York, was of a character which could not have occurred in a masonry or concrete dock. There is ground, therefore, for adding the cost of repairs upon its structure to the above amount, which will give a grand total of $612 022 expended in repairs, during a comparatively brief period, to the timber docks proper of the navy.

Published Comparisons of the Cost of Maintenance of Stone and Timber Dry Docks.—Efforts have been made from time to time in testimony before Congressional Committees, in the public press and in pamphlet literature, to show that the cost of maintenance of dry docks of timber is much less than that of docks of stone. In 1897 the Simpson Dry Dock Company, of New York, issued such a pamphlet, which, since it is a public argument for timber and against masonry as the material of national and other dry docks, may be properly discussed herein. The pamphlet states that "the estimated cost of repairs to all of the docks is based upon close observations and our experience of over forty years." Estimates, it may be said, are not facts; and it would appear that the absence of exact knowledge as to the cost of maintenance of the various national docks has led, in this pamphlet, to deceptive conclusions. It should, as well, be repeated emphatically here that a just comparison of the maintenance of the two classes of docks can be based only on the cost of repairs to the dock structure proper, excluding all accessories, such as caissons and pumping plants.

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The Simpson pamphlet estimates the total cost of repairs to the Stone Dry Dock at the Navy Yard, New York, from its completion to 1897, as $250 000, an average of $5 434 per year. Excluding accessory structures, the actual figures, as already given, are $210 618, or $4 680 per year.

Again, there is given a total of $16 000 for general repairs to Timber Dry Dock No. 2, at the Navy Yard, New York, an average of $923 per year. The facts are that the repairs to the dock structure proper were only $1 982, or $270 per year; but, as already shown, the cost of repairs, renewals and reconstruction of the entrance will be $300 000, which will bring the yearly average for repairs to the dock-structure proper up to $30 198. It should be stated, however, that when this work is completed, no further repairs will, probably, be required for a considerable time.

The Simpson figure for general repairs to the Granite Dry Dock at Norfolk since the year 1865, is $125 000, or $3 906 per year. It will be observed that this is not claimed as an actual, but as an estimated, expenditure. In the Government report, of the Board on Dry Docks, of 1898, the total repairs from 1882 to 1897, only, are given. It is therefore to be presumed that there are no data available as to expenditures during prior years. This report gives a total outlay, for these 15 years, of $61 430, of which only $18 776 were applied to the structure proper, a yearly average of but $1 251. The Simpson estimate is from the year 1865, and it is a matter of history that this dock and its gate were seriously injured by the Confederates during the Civil War. Large amounts were spent in repairing the damage caused by their attempt to blow up the dock, and, even if the Simpson estimate be received as correct, it is apparent that the bulk of the expenditures must have been unusual and not due to ordinary wear and tear.

With regard to the Timber Dry Dock at the Navy Yard, Norfolk, the Simpson Company estimate the repairs to the dock structure, up to the year 1897, at $6 000, or an average of $818 per year. The expenditures given by the Government Dry Dock Board must, however, be regarded as authentic. These show, on analysis, that during this period the repairs to the structure proper of this dock amounted to $13 576, or an average yearly cost of $1 697, and when, in the current year, the repairs appropriated for and now being made are completed, the average annual expenditure will have been $4 857 upon the dry dock structure proper.

The Granite Dry Dock at the Boston Navy Yard is estimated, in the Simpson pamphlet, to have cost for general repairs, from its completion to 1897, $150 000, or a yearly average of $2 343. The Dry Dock Board gives the total expenditures from 1866 to 1897 as about $131 000, of which one-half was upon the dry dock structure proper, making an average yearly expenditure upon the latter of about $2 000.

Mr. Cathcart. No precise record seems to have been available giving the cost of repairs in detail prior to 1866; but the Board states that during the years 1855 and 1857 about $21 000 were expended upon new engines. It is more than probable that the repairs to the dry dock structure proper, from the time of its completion down to 1866, were very slight, since the annual report of the Chief of the Bureau of Yards and Docks for the year 1853 states that the stone dry dock at Norfolk had then been in use for twenty years, and was in perfect condition.

It would appear, therefore, that at least $21 000 of the general repairs estimated in the Simpson pamphlet at $150 000, must have been applied to the pumping plant prior to 1866, and, if this estimated total of general repairs is correct, the average expenditures upon the dry dock structure proper, from its erection to the year 1897, must have been very much less than $2 000 per year, probably about half that amount, since the Dry Dock Board gives about $65 500 as the expenditures upon accessories from 1866 to 1897, and a further amount of $21 000 as spent upon new engines in 1855 to 1857. These two items make a total of $86 500, which leaves but $63 500 of the Simpson estimate for the maintenance of the structure proper during about 65 years, an average of less than $1 000 per year.

In referring to the concrete and granite dry dock at Mare Island, Cal., the Simpson Company began their estimate of repairs from the year 1891. From this time to 1896, they give the general repairs as $25 000; to this they add $50 000 for a new caisson—which was a duplicate and not a replacement—obtaining thus, for general repairs, a most erroneous total of $75 000, with an absurd yearly average of $12 500. As a secondary proposition, they state, that, if the item of the caisson be omitted, the repairs will average $4 166 per year, and add, finally, "this is claimed to be a modern, stone dry dock in every respect."

These estimates will now be studied in the light of the facts. A statement, as to the actual expenditures upon this structure and its accessories, obtained upon inquiry, gives the total for repairs, from the dock's completion in 1891 to 1897 inclusive, as only $14 406.86, of which amount $8 258.55 were upon the pumping plant, $4 534.59 upon the caisson, etc., and but $1 613.72 upon the dock proper, an average of $230.53 per year during this period of 7 years. The average expenditure for general repairs was $2 058.12, which figure is just half of the corresponding Simpson estimate.

The cost of the caisson mentioned has no bearing either on facts or estimates as to repairs. It is a duplicate, built as a relief for that within the dock, and its cost, $50 000, belongs to equipment, not to repairs. There is no place at or near Mare Island where an injured caisson could be docked; and, in view of this isolated situation, the Navy Department decided to have the duplicate built.

The erroneous estimate of $25 000 for general repairs, as given above, can be accounted for only by the supposition that there were included therein certain expenditures made in 1896 for a 40-ton crane and track for use about the dock, which, as with the duplicate caisson, must be classed as equipment, not repairs. Mr. Cathcart.

It will be seen, therefore, that the repairs to this "modern stone dock" were practically insignificant, in respect of the structure proper and exclusive of the accessories, which are of a class belonging to all dry docks.

The Simpson statement as to the timber dock at the Navy Yard, League Island, gives an amount of $10 000 for repairs and improvements from March 4th, 1891, to January, 1897, a period of about 6 years. By deducting $4 000 for raising the coping and $5 000 for repairs caused by the method of carrying out the work upon an adjacent wharf, the cost of maintenance of the dock is reduced to a total of $1 000, and an average of $166.67 per year.

As has been shown previously, Congress has appropriated $64 000 for repairs to be made during the present year to the structure proper of this dock. It is scarcely necessary, therefore, to challenge the estimate as above, since this appropriation alone gives a yearly average of over $7 000 for repairs to the dock proper.

The timber dry dock at Port Royal, S. C., was not built by the Simpson Dry Dock Company. Their pamphlet gives an estimate for general repairs to dock and accessories of $5 000 for the 2 years, or $2 500 per year. This yearly average is believed to be substantially correct.

In concluding the analysis of the cost of maintenance of the various national dry docks, as estimated in the pamphlet issued by the Simpson Dry Dock Company, it is desirable to state that this analysis has been undertaken only in view of the fact that this company is foremost among the advocates of timber dry docks, and that its arguments, as published in this pamphlet, and as presented to the Naval Committees of the two Houses, have had much influence in shaping the action of Congress with regard to the material of construction for the national dry docks.

In considering the cost of repairs to timber dry docks, as given in this pamphlet and in other arguments presented by the advocates of modern docks, it will be noted that the timber docks of the Navy, which are cited as examples of the small expense of maintenance for this material, range in age from but 2 to 10 years. Being thus practically new, it is obvious that their deterioration cannot have progressed sufficiently to give a true indication of the cost of repairs in the long run. For a correct estimate as to the latter, the comparison must extend over a period, at the close of which decay shall have reached such a stage that it will be necessary to make repairs sufficiently extensive to

Mr. Cathcart. restore the dock to its original sound condition. In other words, make the comparison, not on repairs for temporary patch-work, but on the reconstruction of a fair portion of such a dock, built, as it is, of perishable material.

Even such comparison as it has been possible to give herein, from the data of these practically new timber docks of the Navy, will show, however, that their yearly cost of maintenance, as presented by the advocates of wooden structures, is misleading and far below a correct measure of this item. These docks are certainly equal, if not superior to, the average of timber dry docks constructed elsewhere; it is not possible to admit that their care is in incompetent hands; and yet, it is clearly apparent that their cost of maintenance is very large, exceeding greatly that given in the pamphlet referred to and in other reports.

The comparison instituted herein has been based upon the cost of repairs to the dry dock structure proper. The advocates of timber construction seem to prefer a comparison founded on the entire general repairs of the docks, including all accessories, such as caisson, pumping plant, etc. It is submitted, that, even if the latter method were proper, the showing which has been presented to the public is unfair, in that the timber docks, being from but 2 to 10 years old, have not reached a time when these accessory structures require renewals or extensive repairs. To obtain correctly this yearly average for general repairs, which timber advocates prefer, this stage of renewals to accessories must be awaited. When it shall have been reached, it is plain that the comparison will be much more unfavorable to the wooden dock.

With regard to the stone docks of the Navy, it must be said, also, that the cost of their maintenance, as presented by their opponents, is largely misleading. The comparisons given, as to these docks, by timber advocates, are founded, not upon the maintenance of the structure proper, but upon the general repairs to the dock and its entire accessories—a principle which is obviously wrong. The errors arising from the adoption of this method are magnified in this case by the very considerable age of the majority of these four docks, two of them being 65, and one 48 years old. The repairs to the accessories form, therefore, a very large proportion of the total cost of maintenance, amounting, with the dock at Norfolk, for 15 years, to 70% of the total; with that at Boston, for 31 years, to 53%; and with that at Mare Island, for 7 years, to 89 per cent. The maintenance of the timber docks, on the other hand, if including repairs of all kinds, has not, because of their youth, comprised any considerable expenditures upon caissons, gates or pumping machinery.

In summation, it may be said that the data set forth herein would appear to show conclusively that the average yearly expenditure for the repair and maintenance of a masonry dock, built in the best manner

and of superior materials, is inconsiderable, both in the aggregate and in the percentage of the original cost; while the same items, with regard to the timber docks of the Navy, are unquestionably very large. Mr. Cathcart.

In view of the fact, that, for 10 years past, the Government has been practically committed to the policy of timber docks, having, within that period, built six of this material, and, of the further fact, that, by recent legislation, three more of these structures are authorized, the subject is presented to the members of this Society for discussion. The growth of the United States as a nation, and the results of the recent war, foreshadow an early and considerable increase in the force afloat. The defense of a coast line, extraordinary in extent; the acquirement of territory beyond the sea, which must be held—as it was won—largely by the Navy; the probable entry of the United States into Asiatic politics; and the prospective revival of the merchant marine, which the fleet must lead and guard—all point to this. The full consideration of any factor, therefore, which is essential to naval strength—as a trustworthy system of dry docks surely is—must be of interest, not only to the engineer, but to every man who has at heart the welfare of the country's gallant fleet.

TABLE No. 1.

Dock.	Years.	Total on dock proper.	Average, dock proper per year.	Previous average per year.
Stone, Boston	31	$62 000	.....	$2 000
Stone, Norfolk	15	18 776	.....	1 251
Stone, Mare Island	7	1 613	.....	230
Totals	53	$82 389	.....	$3 481
Average	17⅔	27 463	$1 558*	1 160
Timber, No. 2, New York	10	$301 982	.....	$30 198
Timber, League Island	9	74 202	.....	8 245
Timber, Norfolk	10	48 576	.....	4 857
Timber, Port Royal	4	16 262	.....	4 065
Totals	33	$441 022	.....	$47 365
Averages	8¼	110 255	$13 364*	11 841

* The item $1 558 is on the basis of three docks at 17⅔ years per dock. Similarly, the item $13 364 is based on four docks at 8¼ years per dock: $13 364 is about 8½ times $1 558; or, taking previous averages, $11 841 is about 10 times $1 160.

Commodore GEORGE W. MELVILLE.*—This subject, in the speaker's opinion, is of prime importance to the further development and maintenance of that strength upon the sea, of which—more and more as the years pass—the nation will find itself in need. Com. Melville.

* Chief of Bureau of Steam Engineering, U. S. N.

Com. Melville. With the acquirement of distant island territory, swarming with bronze-hued aliens, "The White Man's Burden," of which Kipling writes, in the civilization and enlightenment of "the silent, sullen peoples," is laid at last on the broad shoulders of the United States. With this there looms up as well a golden future on the Pacific, in the extension of the nation's trade to its uttermost island and to the vast Asian littoral beyond. Finally, the time seems ripe for the merchant marine to take again its proud place as of old, when its flag streamed over every sea.

These considerations point with no uncertain hand to the nation's need in the early future of a strong force afloat; and, in maintaining the efficiency of that force, a worthy system of dock construction is a cardinal essential. The speaker desires, therefore, to place on record his opinion—as he has done in the past—with that of the Secretary of the Navy, with that of his distinguished colleague, Commodore Endicott, and with that of his predecessor, Admiral Matthews, as being opposed most strongly to the flimsy structures, costly in their weakness, which have been built as timber dry docks for the Navy. As to the use of wood for the docks of private corporations, the speaker has nothing to say. It is plentiful and cheap; the average draught of merchantmen is light; the structures are less costly; and the exigencies of private business may require them. With Government constructions, however, none of these considerations holds. A wise nation builds, not only for its generation, but for its sons of the coming years; and we, in our day, are not true to this principle in the timber-veneered excavations, perishable and perilous, which, for 10 years past, have been inflicted upon the Navy.

The speaker will consider, first, the comparisons which have been advanced by the opponents of stone docks, with regard to the first cost of the two types. In this matter it has been the practice of some timber advocates to compare the cost of wooden docks, built by contract for the Government, with that of granite docks constructed in the navy yards by day's labor, extending over a long series of years, as appropriations were made from time to time by Congress. Thus, it is found that the Simpson Dry Dock Company, in its public arguments, has instituted such comparisons as to first cost between its own timber docks, built by contract for the navy in from 22 months to $2\frac{1}{2}$ years, with stone docks the construction of which by the Government extended, respectively, over 6, 10 and 19 years.

Such comparisons are grossly unfair. The only just system as to this would be to compare timber docks and stone docks built under exactly the same conditions; that is, by contract, after public advertisement, under authority from Congress to enter into contract for the full completion of the docks at the earliest possible date. For the first time in the Navy's history it is now possible to compare the relative

cost of timber and masonry docks of approximately the same size and to be built under substantially the same conditions. Com. Melville.

The proposals opened recently for a dry dock at Boston will furnish a fair comparison in this respect. The lowest bids for this structure show that the Government will be able to build by contract a dock of the very largest size, constructed of concrete and stone masonry, within the sum of $1 000 000, or at an advance of but 25% upon the amount appropriated by Congress for each of the three timber docks now authorized. In view of the cheapness of wood, as compared with concrete and stone, it would appear either that its manufacture into a timber dock is a somewhat costly operation, or else that there is a very large profit included in the sum appropriated by Congress for these timber docks. The matters of first cost and of cost of maintenance, however, are minor issues only, which should have no determinate bearing.

The real question is the stability and safety of a structure which may, at any time, hold within it a great battleship, costing millions of dollars and swarming with hundreds of men. The nation, to save dollars, cannot afford to imperil its naval strength or to risk human life; and these very things its policy of unwisdom as to docks compels daily in the entrance of ships into structures which have shown, time and again, their weakness. None of the timber docks of the Navy exceeds 10 years in age, and yet abutments have moved inward, and other injuries, more or less serious, have occurred to them, extending from the bursting in of the altar system, as at League Island, Port Royal and New York, to the distortion of the floor of the dock, as at Port Royal, and the partial collapse, in the case of Dry Dock No. 3 at New York. That no catastrophe—great in financial loss and appalling in the destruction of life—has yet occurred, is due solely to the care and skill of those in charge of these weak docks. The battleship *Massachusetts* was saved—during the partial collapse of Timber Dock No. 3 at New York—only by the hasty flooding of the structure and the hurried removal of the ship from its treacherous grasp.

In discussing the relative strength of the two types of docks, the opponents of stone lay stress only upon the capacity to withstand the vertical thrust of the ship upon the flooring of the dock. The stone dock may rest on a bed of natural rock, or in one of impervious clay, or upon piling foundations. The timber dock is built always upon piles. The calculations referred to, therefore, in the most difficult cases, consist simply of those based upon the strength, penetration and cross-section of the piling, involving but little engineering skill. These statements are *ex parte* and fallacious, and ignore wholly the inherent and vital weakness of the timber dock, which lies in its tendency to inward collapse under the hydrostatic pressure horizontally

Com. Melville. and from without, upon its sides, and vertically from beneath, upon its floor.

No such danger threatens a properly constructed dry dock of masonry. With good materials and workmanship, it is possible to make it absolutely water-tight, and the structure has, in itself, a weight more than sufficient to withstand the hydrostatic pressure from without. Stone and concrete give it that weight, and, since these materials, when properly embodied, will last for centuries, that weight can never be lost. For all practical purposes, the stone dock is as enduring as the everlasting hills.

With the timber dock the case is radically different. Its integrity —fleeting, at best—depends solely upon success in pinning it to the ground. Such a dock has merely a thin veneering of timber, lining the excavation and held down by piles driven into the soil, to which piles the timbers are bolted. It would be, obviously, both futile and dangerous to attempt to exclude from such a structure the entrained water in the surrounding soil. The timber dock, therefore, is one of tight joints only in the abutments, being open as a sieve elsewhere, and leaking, more or less, as the amount of water in the circumjacent ground is large or small. It lacks utterly, therefore, the tightness and weight which give stability and permanence to the dock of stone.

Furthermore, it is apparent that this hydrostatic pressure—which, in graving docks, is the chief force to be met and guarded against—increases with the depth. As this dimension increases, therefore, the more unfavorable become the conditions for the wooden dock, which has relatively no inherent weight to oppose resistance to pressure from without. With such increase, its stability and safety suffer rapid decrease, and there is swift growth in the tendency to rupture the bottom and sides. From this point of view, the question becomes one of great importance and risk, with regard to the very deep docks which are now necessary for the larger cruisers and battleships of the Navy.

It has been shown that the leakage of the timber dock depends simply upon the amount of water present in the adjacent soil. Now, this very leakage, which in one sense is an element of safety to the structure, is, in another, a source, ultimately, of sure and grave danger; for soil is carried in with the leakage, the latter increases, and there is displaced finally so much material as to cause at least a partial failure. This was the case recently in Dry Dock No. 3, at the Navy Yard, New York.

In a stone dock, however, a leak is not followed by such results, since the structure is homogeneous and all joints are water tight. The failure of one or more of the latter does not, therefore, involve necessarily the failure of others. As a matter of fact, the stone docks at the Navy Yards of Boston and New York have leaks to-day which have

existed for many years without measurable increase and without involving the structures to any extent. It is true that the masonry at the entrance of the Boston dock has been displaced slightly by the action of frost upon the open joints; but the weight of the structure is such that, if these joints were all open, it would still act as a dry retaining wall or arch, without danger of failure. Com. Melville.

At least two of the timber dry docks of the Navy suffer from excessive leakage, with its resultant dangers. At the Norfolk yard, the wooden dock is founded in a bed of clay of excellent quality, and the leakage is very small. That of Timber Dock No. 2, at the Navy Yard, New York, amounts to little more than 100 galls. per minute, which is moderate, considering the character of the soil. The leakage of the timber dock at Puget Sound, is not much more; but this is due chiefly to the fact, that, in the construction, provision was made for admitting the water from the soil, through a system of sub-drains and valves, into the drainage culvert, and thence to the pump-well, from which it is discharged by the drainage pumps. This was done in order to relieve, largely, the bottom of the dock from hydrostatic pressure.

With the wooden dock at League Island, however, no such advantageous conditions prevail. It has always leaked excessively, the amount entering from the surrounding soil varying from 1 200 to 1 700 galls. per minute. This great volume taxes fully the drainage apparatus originally installed; and, to obviate the inconvenience and risk, which would arise from the failure of this pump, a duplicate apparatus has been erected. This duplicate pump stands to-day an eloquent, if silent, monitor as to the unwisdom of the wooden dock for national use. When a ship is in the dock and it is necessary to keep the floor dry, one of these drainage pumps runs constantly.

At the Port Royal Dry Dock, the leakage is about the same, and it was found necessary to fit reliefs very largely in the bottom and sides, in order to prevent a recurrence of the upheaval and rupture of the structure, which occurred about the time of its completion.

The foregoing has referred to the stability and safety of national docks of wood, in the past. The speaker will now examine briefly the programme for the future. In the Act of Congress, approved May 4th, 1898, there were authorized four new dry docks for the Navy, one each at Portsmouth, N. H.; Boston, League Island and Mare Island—all to be of the largest size. All these docks were to be of timber, at a cost not exceeding $825 000 each; but the Secretary of the Navy was authorized, in his discretion, to build one of granite, or concrete faced with granite, the limit of cost, in this case, being increased by $200 000.

In the exercise of the discretionary powers lodged in him, the Secretary of the Navy decided to build the dock at Boston of granite, or concrete faced with granite. As stated previously, the bids for this

Com. Melville. dock fall considerably within the appropriation and are about 25% in excess of the amount appropriated for each of the wooden docks.

The absurdity of a further continuance of the policy of extravagance and disaster in the building of wooden docks is shown, not only by the facts above given as to the relative costs of the types, but by further facts with regard to the soil in which the proposed docks are to be located. For instance, that to be built at League Island will meet the same adverse conditions to which its timber predecessor there has succumbed. The soil there contains a stratum of compact sand and gravel, which is water-bearing, and the surface-water, when it meets this stratum, flows into the dock, the latter acting as a huge drain. Shall there be erected at League Island, more duplicate pumps, as costly monuments to the timber folly?

Again, borings at the possible sites for the proposed timber dock at Portsmouth, N. H., show, thus far, that the excavation for this dock must be blasted from granite, and, then to comply with law, be lined with wood. The advocates of timber could ask no deeper devotion to their theories and interests, than this.

In the speaker's view, Congress has acted in this matter without full knowledge of all that bears upon the case. Commodore Endicott, in a report to the Secretary of the Navy, which has been transmitted to Congress, has set forth the facts with clearness and great strength. The matter is of such importance to the future of our gallant fleet, that its discussion cannot be too wide. As an engineer who has served both on land and sea, the speaker regards it, therefore, as a duty and a pleasure to present his views on this subject.

Com. Endicott. M. T. ENDICOTT,* M. Am. Soc. C. E.—It is probable that the first dry docks were constructed in Great Britain. It is known that, in the time of Queen Elizabeth, docks for the reception of vessels needing repairs were made by excavations into the soil bordering water-ways, lined with timber and backed with clay to protect the floor and sides, and fitted with gates at the entrance to shut out the water. The vessels at that time were quite small and the docks were of corresponding dimensions.

In the time of Charles II several docks were constructed in English ports, for the use of the British Navy, which were of considerable size. These were also of timber.

So far as the records now show, all the earliest docks were of wood, and were what are now called graving docks; that is, the dock was formed by making an excavation in the earth and lining the bottom and sides, thus forming a receptacle for the vessel, into which it could be floated, landed upon supports, and left accessible for examination and repairs when the water should be exhausted from the dock. As time went on, and larger and better docks were called for, the con-

* Chief of Bureau of Yards and Docks, U. S. N.

struction gradually changed, and brick and stone came into service for the construction of certain portions of the docks, and at present there are on the Thames many docks which are built of all these materials in combination; that is, partly of wood and partly of brick or stone, and, in the case of those of later construction, of concrete, also. Com. Endicott.

The transformation has progressed in Europe to such an extent that, at the present time, practically all the large docks are built exclusively of masonry, classifying concrete, for the present purpose, under the head of masonry, although strictly speaking it is not. The records, so far as accessible, show that abroad there are no constructions of this kind in timber. The present Chief Constructor of the Navy, Mr. Hichborn, who visited Europe in 1885, states that at that time the later docks were built largely of concrete, and were of the most substantial construction; and speaking of those on the Thames (private docks), a number were built of wood, but the principal docks were of masonry or concrete.

Masonry docks are the outgrowth of some centuries of experience in the construction and use of works of this kind. In foreign practice, it is a case of the survival of the fittest.

The first dry docks in America were constructed by the United States Government, and were begun about the year 1828. Before these were undertaken, Congress authorized the Secretary of the Navy to engage a skilful engineer to prepare the designs for these structures, and Colonel Loammi Baldwin, of Boston, Mass., was employed. He came to the duty well fitted for the work, having prepared himself especially by a visit to Europe for the purpose of studying structures of this class. With the experience of the Old World before him, he very wisely decided to model the structures for this Government after those which had been evolved from the experience of so many years abroad, and he made the two docks then authorized for the Navy, at Boston and Norfolk, the finest examples of fine masonry in submarine works that then existed in this country. They were regarded as among the best and most creditable public works built for the Government up to that time, and were pointed to with pride as works showing great skill in design and execution, and well adapted for the purpose of docking and repairing the vessels of the Navy.

The construction of these docks extended over several years, and they were not completed until 1834. They were built at a comparatively moderate cost, the Boston dock costing about $677 000,* and the Norfolk dock about $943 000. Both were substantially founded upon piles, were built entirely of stone, and were faced with cut granite of the finest quality and workmanship.

The granite dry dock at the Brooklyn Navy Yard, completed in 1854, followed, and, for various reasons, was a very expensive structure,

* This dock was afterward extended in length, bringing its cost, to date, up to $972 000.

Com. Endicott. costing $2 000 000; and then came the dry dock at Mare Island, Cal., which is a concrete structure lined with cut granite.

In the interval between the construction of the Boston and Norfolk docks, and that at Mare Island, the Government built several floating dry docks of wood. Two or three were of the balance type, that is, of one section, and several others were of the sectional type, and in the case of those 300 ft. or more in length, the sections numbered as many as nine. The balance docks were used in connection with shallow basins, as were the sectional docks, in order that a ship, taken up by the floating dock, could, by the sinking of the dock upon the floor of the basin, be hauled out upon a railway on shore for extensive repairs, while the dock could be used for docking vessels for slight repairs, not requiring hauling out.

It is not the purpose to discuss the merits of floating docks. This is simply mentioned as an incident in the history of the dry docks of the Navy. They had their day, but not proving so satisfactory, durable or convenient as the masonry graving dock, they were not replaced. They gradually deteriorated so much as to require extensive rebuilding. To-day only one of these docks remains in service, that at the Portsmouth, N. H., Navy Yard, and it is now safe only for vessels weighing a little more than 1 000 tons.

The timber graving docks of the present day in the United States did not originate in the Navy. They were constructed first on the Great Lakes. A timber dock built at Buffalo, N. Y., as early as 1840, was inspected by the speaker a few years ago, when he was assured by the officers of the company then owning it, that it was built not later than the year named. The speaker has been led to believe, from some investigations made since, that it was built even earlier than that time, and that the timber graving dock made its appearance in this country as early as 1837 or 1838. Others followed, and many docks of this type are now to be found in Buffalo, Cleveland, Chicago, Detroit, Port Huron and other places. While they differ in details from each other and from those built on the Atlantic coast, they are all of the same general type, in that the excavation is lined entirely with wood secured to piles driven in the bottom and upon the slopes of the sides, and faced with longitudinal timbers, forming steps or altars upon the sides.

The first timber dry dock built upon the Atlantic Coast was at Boston, Mass., in 1854, fully 14 years after the construction of a similar dock upon the lakes. It was of the same general construction, and differed only in details. Other docks upon the Atlantic Coast followed, all built by J. E. Simpson & Company, so that here this type of dock has come to be called the "Simpson" dry dock.

About the year 1886 the question of the construction of additional dry docks for the Navy became an important one, as the Navy was

building vessels of steel which required frequent docking; and, as the old sectional docks were disappearing, the need for further docking facilities was felt. Com. Endicott.

Without entering into all the reasons which led the Government to abandon the construction of masonry docks and take up with those of timber, it may be said that this was due to the advocacy of this type of dock by persons in civil life who were interested in their construction, and by a few officers inside the naval service who had championed the cause of this type.

The timber dock was presented to the attention of the Navy as possessing not only many good qualities, but many far superior to those of the stone dock. The most attractive arguments presented by the advocates of the former were its cheapness, its convenience in the way of accessibility to the floor by the gently sloping sides and low altars, its width on top, admitting light and air freely, its immunity from the disintegrating effects of frost, and its greater comfort for workmen in winter weather. A negative advantage, as urged recently by one of its strongest advocates, before the Senate Naval Committee, is that "it will last 40 years, and that is long enough for anything to last."

The arguments advanced by its advocates were successful in securing favorable action in the committees of Congress, and the first appropriations for timber dry docks were made in 1887, providing for two, one at the New York yard and one at the Norfolk yard. These were followed by one at the League Island yard, completed in 1891; one at the Port Royal Naval Station, completed in 1895; one at the Puget Sound Naval Station, with a masonry entrance, which was completed in 1896, and one in the New York Navy Yard, called Dry Dock No. 3, completed in 1897. The Government was thus fairly launched on the policy of the construction of timber dry docks, and it seemed as though it would be impossible to again secure an appropriation from Congress of an amount sufficient to build a masonry dry dock; in fact, the acts making appropriation, in almost every instance, as well as limiting to an amount only sufficient for a timber dry dock, provided specifically for one of such material.

The last Congress made provision for four timber dry docks, to be constructed, one each at Portsmouth, N. H., Boston, Mass., League Island, Pa., and Mare Island, Cal., but provided that the Secretary of the Navy might, in his discretion, build one of them of granite, or concrete lined with granite, and in such case the limit of cost was to be increased $200 000, to a total of $1 025 000. The Secretary exercised the authority granted by this act by deciding that the Boston dock should be of granite or concrete lined with granite, and plans and specifications were prepared for a dock of this character. Bids were opened on January 31st, 1899, in the Bureau of Yards and Docks, and resulted in a proposition providing for a dock with granite facing and

Com. Endicott. concrete backing, complete, for about the amount of the appropriation, or about $1 000 000. This result was much more favorable than anticipated, and showed for the first time that a masonry dock of the largest size and the best construction could be built for a sum not greatly in excess of the cost of a timber dock of the same general features and dimensions.

Prior to the final passage of the act making appropriation for these last-named docks, the Department had been before the Senate Naval Committee in advocacy of the construction of granite rather than timber docks, and the Board on Dry Docks, appointed by the Department in 1897, had reported recommendations which were, in effect, in favor of the policy of docks of masonry instead of timber. This report was laid before Congress by the Department. The Department had thus taken such steps as it might to bring to the attention of Congress its views and advice in the matter of the construction of these important structures, and the action of Congress, having been substantially adverse to this advice, the Department was not disposed to pursue the matter further with respect to the docks then authorized and provided for.

The possibilities in the way of the construction of masonry docks, as shown by the letting just referred to, were of such importance that the Bureau believed it advisable to bring them particularly to the attention of the Department, and, through it, to Congress, with a view to securing a reversal of the action already taken in the act referred to.

The subject of the advisability of constructing these works of stone rather than of timber had received much attention from engineers and from the press, and there seemed to be a wide interest in the matter, and a strong public sentiment against the course prescribed by Congress with reference to the material of which they were to be constructed. It had also become quite apparent that the general sentiment among officers of the naval service who took an interest in the matter and had paid much attention to it, was, with very few exceptions, strongly in favor of masonry docks. Altogether, it seemed to the Bureau that the result of the bids for the Boston dock indicated that the time had arrived for the Department to make another effort to save the Navy from the construction of more timber docks. In making such a move there seemed to be a prospect of success, and the Bureau addressed a letter (published in the New York *Tribune*), to the Secretary of the Navy, strongly urging the Department to make another effort with Congress to abandon the construction of timber dry docks and re-enter upon the policy, which it instituted in the early years of the century, of building for its Navy the safest and best dry docks, and which had become the policy of all other first-class naval powers.

In pursuance of this recommendation, the Department has again

brought the matter to the attention of Congress, through its naval committees, and has laid before them the Bureau's recommendations in the matter, with its earnest approval. The matter has been brought up at an opportune time, for several reasons: Because the timber docks heretofore built are beginning to show that they are temporary and unsafe structures; because the proposals for the Boston dock show that a first-class masonry dock can be constructed for a reasonable sum and at a moderate increase over the cost of a timber dock; and because the public mind is alive to the subject and strongly in favor of good, substantial, creditable structures of this kind for the Navy. Therefore, the recommendations and efforts of the Navy Department, backed by a disinterested, logical and earnest sentiment, are almost sure to bring about a change in the present policy. Com. Endicott.

A timber dock is a temporary structure, and as the chief material entering into its construction is wood, it requires very extensive renewals at an early day, varying, according to the kind of wood used and the local influences, from 20 to 25 years, when they amount to a practical rebuilding of the upper portion of the structure, and frequently its entire reconstruction. While the first cost of a timber dock is small, compared with that of a masonry dock, the rapid deterioration referred to results in considerable expenditures at the end of a few years, if the structure is kept in good condition, and if the repairs and renewals are of an extent necessary to preserve the dock in a condition as safe and good as when completed. The experiences in the Navy, thus far, with timber docks, although extending over only a few years —the dock now longest in use having been built about 10 years ago— show that the expenditures necessary to restore them to their original condition at the end of this latter period are very considerable, and appear to be several times as great as those for a stone dock of about the same dimensions. For the purposes of a fair comparison the cost of the maintenance and renewal of accessory structures, as caissons, pumping machinery, etc., should be neglected, and a parallel should be drawn entirely between the expenditures made upon the dry dock structures proper; which, in the case of one, would be stone and concrete, and in the case of the other, timber.

The records of the stone docks of the Navy, as far as they have been collated, show that, generally speaking, the repairs are very small indeed; the only instance in which any individual repair has been of considerable magnitude being in the case of the stone dock at the Brooklyn Navy Yard, where the masonry joints became injured on account of the presence of springs and the action of frost. This condition was due largely to the fact that the cement used in the mortar for the joints was native hydraulic cement, not Portland cement. The work was repaired about the year 1886, the masonry at the entrance and some at the head being taken up and relaid with Portland cement

Com. Endicott. of the first quality. Although these repairs have been completed for many years, they appear to be entirely successful, and there has been no recurrence of injury to the joints. It seems quite safe to say that if high-grade Portland cement mortar had been used originally, there would have been no such trouble, and the work would probably have lasted until the present time without any material injury.

The records of the repairs to the stone docks extend over a great many years prior to the Civil War, and the collection of exact and full data in regard to the expenditures upon them is difficult, but those which have been examined and compiled are sufficiently extensive and accurate to be a fair index of the repairs to such structures.

Without repeating here the details and figures, it is to be said that the average annual cost of repairs upon the masonry docks has been very small indeed, and has amounted to a very trifling percentage of the cost of the work.

The records of the expenditures upon the timber docks and their present physical condition are full, exact and easily obtained, and show that not only is the cost of repairs per year very much greater than for stone docks, but that the percentage of the original cost put out in repairs is several times as great as in the case of the masonry docks.

In the matter of cost, the experience of the Navy has been that in the case of the masonry docks last built, it is large; but that those first built, comprising the docks at the Boston and Norfolk Navy Yards, although of the finest quality of cut granite, did not cost as much, being in each case less than $1 000 000. The large cost of the masonry docks was due to several causes; in the first place, they were not built by contract, but by day's work by navy-yard employees, and the materials were purchased or contracted for piece-meal. The appropriations for the work were made by Congress in such a way that labor and expenditures extended over long periods of time, thus enhancing the cost and resulting in decay and injury to the coffer-dams and accessory structures, and the disintegration of the soil exposed in excavations from the action of the elements, on account of the many seasons covered by the work.

The timber docks for the Navy were all built by contract, after public advertisement, and, by the authority of Congress, contracts were made for the entire completion, thus enabling the work to be taken in hand and pushed through with the greatest possible despatch. It will be seen, therefore, that the records of the Navy, in the matter of the costs of timber and stone docks as built, afford no fair basis of comparison, and one of the great advantages which has been urged in favor of the timber dock is the smaller relative cost. It has even been claimed that the interest on the cost of a masonry dock was sufficient to build and maintain a timber dock indefinitely. This comparison

has almost invariably been drawn between a timber dock built by contract, and either the New York or Mare Island stone docks, which latter were built by day's labor, the work extending through a long term of years. Com. Endicott.

An opportunity has been afforded recently for instituting a somewhat accurate comparison of the relative cost of these two types of dry dock by the proposals received for the construction of a concrete and stone dock designed for the Boston Navy Yard. These bids were based upon conditions very similar to those which obtained in the cases of the timber dry docks; that is, Congress having authorized the entire construction of the dock within a certain limit of cost, the plans and specifications were made and competitive bids were invited, after public advertisement, the work to be completed in a period not exceeding 2½ years, and as much shorter as the contractor may desire to make it.

These bids show that a dock of the largest size can be built of concrete lined with cut granite, and complete with all accessories, at a cost of about $1 000 000, or only about 25% in excess of the amount fixed by Congress as a limit of cost for each one of the timber dry docks to be built of the same size and under similar conditions. It will be seen, and must be admitted, that such a comparison is fairer and much more accurate than that previously referred to, where the cost of the timber dock by contract was about $600 000, and the cost of the stone docks at Mare Island and New York was $2 000 000 in one instance and $2 750 000 in the other.

This is the first time in the history of dry dock construction in the Navy that such a comparison of the cost, under similar conditions, has been possible. The result is, of course, beyond dispute, and so favorable to the masonry dock, in the matter of relative cost, as to be of the greatest value and significance. It shows that the relative costs of these works heretofore claimed by the advocates of the timber dock, and their general acceptance by the public, were wrong and misleading.

The claims set up for the timber dock, based upon these comparisons, have had great influence upon the public mind and upon Congress and its committees, and have resulted in the practical adoption of this type of dry dock for the United States Navy for the last twelve years, during which time the Government has more than doubled its docking facilities by the construction of six timber docks. While other claims have been made for the timber dock as superior to the stone dock, there is no doubt that the matter of first cost has largely determined the action of the committees in Congress; but the result foreshadowed in the proposals for the Boston dock is a permanent masonry dock at an increased cost of only about 25% over a temporary dock of timber, which would require extensive rebuilding in

Com. Endicott. 20 or 25 years, and also a small annual outlay for maintenance as against a very large one.

In the long run, the timber dock is much more expensive than the masonry dock. A masonry dock built by contract, after public competition, can be built in about the same time as a timber dock, at a moderate increase of cost and a small annual outlay thereafter for its maintenance, as against a timber dock 25 or 30% cheaper in first cost with a large annual expenditure for maintenance, amounting to a practical renewal every 40 years. The result shows that a masonry dock is cheaper than one of timber.

In weighing the relative merits of the stone and timber docks, in the matter of convenience for use, it can be stated that the conditions are practically the same. A timber dock, as usually constructed, is of great width at the top, which has resulted from an effort to construct the sides of the dock at the least expense and with the least danger of deformation, and the slope generally adopted is that natural to the soil, so as to avoid any considerable thrust from the earth tending to force in the sides. The side construction consists almost invariably of timbers superimposed in such a way as to form altars or steps from the bottom to the top. This makes the entire side construction practically a stairway; and the dock, being wide at the top, admits light and air freely.

Stone dry docks, as generally constructed, are not so wide at the top, and the altars or steps are of greater height. Stairways for ingress and egress are introduced at convenient distances. It has been put forth as a great advantage of a timber dock over one of masonry, that, owing to the form, the timber dock is much more convenient for access and admits more light and ventilation, thus facilitating the work in the dock. The materials of which a masonry dock is constructed lend themselves to any form, and one can be constructed of precisely the same outline as the ordinary timber dock. The general form or outline of the masonry dock is one which is almost universally followed in the construction, and, although less broad at the top than the timber dock, such construction results in no inconvenience or disadvantage to the work carried on in the dock, else the form would long ago have been changed to one more nearly approaching that of a timber dock. In the United States, the last masonry dock constructed, that at Mare Island, approximates very closely to the cross-section of a timber dock, and is a sufficient answer to the claim of the superiority of the timber dock in this respect.

It is to be observed, in this connection, that a dock very wide at the coping level is not without its disadvantages, in that it necessitates cranes, or appliances for handling materials, with very long arms to reach the center of the dock, and this has been held by European builders to be an objection to very wide docks. As a matter of fact, it

may be stated that the masonry dock of the ordinary form admits of all the light and ventilation necessary with a ship in dock. There is nothing to obstruct except the vessel itself, which will, in this form of dock, intercept considerable light, and render the portion of the dock directly under the vessel less light than other portions. It is not known that any difficulty has ever been experienced in any masonry dock through lack of sufficient light and ventilation. The only condition in which there would be any lack of light and ventilation would be in the case where the vessel contained in the dock was of such size, as to practically fill it. Where the cross-section of the vessel approximates that of the dock, this condition could only exist amidships, as the section decreases both ways from the center of the vessel. Com. Endicott.

In the matter of accessibility, while the timber dock affords a convenient stairway at any point, a stone dock admits of the installation of stairways at any intervals deemed necessary or desirable. It can be stated, therefore, with reference to the form of the docks, that the masonry dock meets substantially all the requirements of accessibility, light and ventilation, and, in any event, admits of the construction of precisely the same form or outline as a timber dock.

In the matter of the stability of the two classes of dock, the conditions to which they are subject are practically the same. The ability to sustain the weight of the vessel is met equally by the two, generally speaking. The condition involving the most danger to a dry dock, however, is not the weight of the vessel which it receives, but the pressure of the water contained in the soil below the level of high tide, which acts normally to the exterior of the dock, tending to lift the bottom and force in the sides. It needs little explanation to show that the dock which is capable of meeting this condition more successfully is the one of masonry. The timber dock is built of a comparatively light material, wood. Its weight is a very small element of stability, and its integrity depends upon the measure of success with which the access of water through the soil to the sides of the dock is prevented, and with which the wood, which constitutes the interior lining of the excavation, is pinned or secured to the soil underneath. This is effected by driving piles into the soil, to which the interior woodwork is secured by bolts. It will be seen that such a structure is not well designed for resisting the inward hydrostatic pressure, and in all timber dry docks it is the practice, particularly in those of considerable depth, to provide valves or inlets, through which the water in the soil may find access to the interior of the dock, and be carried away through the culverts to the pump-well, and thence be discharged into the harbor. It has been well said that the only way to preserve the stability of a timber dry dock is to let the water into the dock and then pump it out. The pressure of the water is particularly felt in the entrance works of the timber dock, which, being

Com. Endicott

near the harbor, are subjected to greater hydrostatic pressure, where the slopes are much steeper, and to the water pressure upon the sides is added a great deal of thrust from the soil. It has been frequently found in the timber dry docks that they have been incapable of successfully resisting this force, and in many instances the altars have been forced off, the sides of the entrance works have been forced inward to some extent, the bottom of the dock has been uplifted, and, where defects in the work have existed, an inflood of water, once set up, has resulted very disastrously, and endangered the entire structure. In a shallow timber dock this danger is much less, but in those of great depth, which are necessary for the accommodation of large battleships and cruisers, it is a very formidable one and is ever present.

Leaks in a timber dry dock, particularly a deep one, are looked upon with grave suspicion and require constant watchfulness, because, once started, they are particularly dangerous in a dock of this kind, as its structure, not being watertight, silt and soil may be carried in, undermining portions of the structure, and loosening its hold upon the soil to such an extent as to cause collapse or fear of collapse. This was the experience in the case of Dry Dock No. 3 at the New York Navy Yard.

In a masonry dock the structure is designed to resist by its own weight the forces referred to, and, as its construction is watertight, it embodies all the elements of stability and safety. Any leak which may occur in a structure of this kind, due to a faulty joint, is not attended with the same danger as in the case of a timber dock. The early stone docks, now 65 years old, have had for a great many years some leaks through faulty joints, but these have existed without any material extension of the trouble, and without involving any instability or danger; and these docks have preserved their integrity and usefulness without failure or interruption of any kind.

It is of the utmost importance that a dry dock be absolutely safe. Its collapse might prove to be exceedingly disastrous to property and to life. A valuable war vessel, costing millions of dollars and containing hundreds of lives, should be placed in no dock which is not absolutely safe in every respect, and about which there is no doubt. The feeling of security in placing a vessel in a dry dock under such circumstances is worth the expenditure of any amount of money necessary to provide the very highest type. Accidents of more or less serious character have occurred in the timber docks, but none has ever yet developed in a masonry dock, and the advantages of the latter, in the matter of stability and safety, are immeasurably greater than in the case of a timber dock.

A dry dock is a great public work and one of great utility, and should be, like other works constructed by the United States Govern-

ment, of the best and most substantial quality, and an example, in every respect, of the best practice in constructions of its class. In its material the Navy is not content with anything which is not the equal of the same all the world over. All other first-class powers of the world are building graving docks of masonry of the best, most substantial and enduring type. Is it left for this Government to content itself with cheap, temporary structures, unstable, or embodying the elements of danger? Com. Endicott.

R. G. Packard, M. Am. Soc. C. E.—The comparisons made thus far have been between the timber dry docks of the present day and the granite dry docks of many years ago. In such locations as League Island and Beaufort, S. C., especially, the present concrete structures, made of high-class cements, would be much more satisfactory than the timber docks, or even the old masonry docks that have stood so well, the record of which has been given by Messrs. Cathcart and Endicott. It would be interesting to know the number of days the docks have been in working order, and the cost of repairs per foot of length, and per foot of draught. In other words, how much per ton does it cost the Government for dockage? The comparison of cost made by Mr. Endicott is based on the highest class of concrete, and with more or less granite lining of high cost, and highly finished. Such linings may be a necessary feature, but, in making massive structures, such as these concrete basins, a less costly class of materials could be used, more or less saving might be effected, and an equally good structure secured. Thus the relative costs of timber and concrete structures might be brought nearer together than they are to-day. Mr. Packard.

P. C. Asserson,* M. Am. Soc. C. E.—The speaker's experience in repairing stone docks and in repairing and building wooden docks has been quite extensive. His first experience in repairing stone docks was at Norfolk, Va., where the entrance work had been destroyed by the explosion of gunpowder. The cost of repairing that dock was not very great. The greatest expense was incurred in rebuilding the caisson, which had been destroyed by fire when the yard was evacuated by the Confederates, because, although the inner abutment was destroyed, the outer sill was not damaged, and the repairing of the stone abutment required no great engineering science. It took three years, but the appropriations were small. The speaker's next experience was in rebuilding the abutments and outer works of the stone dock at New York, which had been disintegrated by the water finding entrance, principally from the sea. The chief cause of the trouble was bad construction, something rather surprising. It was found when the abutments were taken apart and the masonry taken down to the sill level, that in the abutments proper there was not a single header. The stones were all stringers, the same as one Mr. Asserson.

* Civil Engineer, U. S. N.

Mr. Asserson. brick laid on another, without a heading course. That was probably one of the principal causes of its destruction. The second cause was the poor cement that had been used. Only ordinary Rosendale cement had gone into its construction. A third cause was that someone had attempted to remedy the leaks on top by caulking the joints with lead, thus forcing apart the stone and admitting more water. The speaker believed this to have been a fatal mistake, and thinks the same thing has also occurred in other cases where lead caulking has been used instead of cement. In the repair of the Brooklyn dock it was found also that the sides had settled and split the head of the dock in three parts. At the head of the dock cracks were found which extended from top to bottom on each corner, and in some places the speaker could run his arm into the crack. Of course the water found ingress there, and frost following it helped to displace the masonry. In repairing, these things were all taken into account, and care was taken to put in headers where the masonry had been loosened. The cement used was the best, not that this country afforded, but that could be imported from Germany. The speaker believes that there have been absolutely no other repairs upon that dock since these were completed in 1888. It is in perfect condition and will probably remain so for many years.

In the construction of the wooden dock, No. 2, at Brooklyn, the speaker had some experience—what some engineers would call "a great deal of experience." The basin of the Wallabout is underlaid by what some call quicksand. The speaker calls it mica sand, but does not know that there is much difference. It is very quick, however, and there are "only 90 ft. of it." In places it was found that piles 50 ft. long could be driven very readily and very quickly, but if the hammer was taken off quickly, the pile would follow it up again. To stop the quicksand the contractor resorted to close piling, ballast stone, bags of concrete and all sorts of expedients. Finally, after a great deal of trouble, the concrete was put down, and, by the friction of the sand on the piles, sufficient stability was found in the bottom to hold any ship that the dock would take; but with the sides it was different. The speaker does not think it is generally known, but shortly after the dock was built it was found that in driving home the wedges at the ends of the shores when docking a ship, the dock would go back or spread on both sides as much as 4 or 5 ins. That happens even now whenever a vessel is shored. It is only a question as to how long the diagonal timbers will bear the continual thrust on them. The soil that should pack up to the back of the timbers, and make them steady, is of such a nature that it falls away from the back of the altars and diagonal timbers, leaving the sides of the dock without any support on the back except the piles. The speaker regards the dock as unsafe for docking a heavy ship that has any list.

As long as a ship is perfectly upright and balanced upon the keel blocks, all will be well. Mr. Asserson.

The speaker saw the construction of the timber dock at Newfoundland. The bottom consisted of shale rock, and as piles could not be driven successfully, a concrete bed, 6 ft. thick, was laid down, and formed a very good bottom.

About two years ago the speaker repaired the outer abutments, the sill and the sheathing on the abutments of the timber dry dock at Norfolk, Va., which had been destroyed by the teredo. The cause of this destruction was as follows: When the *Texas* was being built at Norfolk, she was put in the dock to receive her armor, and kept there some nine or ten months. Unfortunately the caisson had been put on the inner sill, thus leaving the outer sill and abutment exposed. During this nine or ten months the teredo completely riddled and demolished the outer side sills as well as the surface planking or timbering of the abutment proper. The sills were completely destroyed from high-water mark to within 1 ft. of the bottom. There, the sill had been covered by mud, and was, of course, in good condition. The covering of the wing walls, extending out on the side, and the end sill of the wing walls were entirely destroyed. These wing walls had been covered by creosoted planking underlaid with ship felting as a protection against the teredo, and that fortunate circumstance had saved the inner plank, but the outer plank was completely gone, and in places had commenced to fall off. In a year or two more the inner planking of the abutment and of the wing walls themselves would have been completely destroyed. By degrees the wing walls and the abutments were stripped, and, by local cofferdams, the whole of the front covering was renewed at a cost of about $10 000. A new sill on each side of the dock was put in and overlaid with a built-up Z-bar of steel, secured in such a way that the sill will probably not be disturbed by the teredo for many years.

The speaker does not know of any chemical preparation with which wood can be treated whereby it will be enabled to completely resist the teredo for more than 10 or 15 years.

The speaker is familiar with the structure of all the docks from New York southward. The timber used in the facing, and subject to decay, has been merchantable timber of good quality, and has been nearly all heart wood, where it was possible to obtain it. Of course, in structures of that kind, where large quantities of timber are used, there will be a little sap wood, and there is no doubt that such wood begins to decay at once. That was the case with the dock in Norfolk, where, two years ago, the speaker commenced to repair the upper works. The piling, the cross-cap timbers and the diagonal bracings were found badly decayed from the coping to the low-water line.

The spreading of the sides of the dock previously mentioned is due

Mr. Asserson. to the difficulty of backing the soil up against the back of the altars when building. Further, being a disturbed soil, on a slope of 35°, it settles away from the altars, and, of course, the space left fills with water. The same trouble does not occur with the stone dock, where the retaining wall is vertical on the back.

The wooden docks have had the same care as the stone docks. Those who have been interested in dock construction for the last 30 years have had great anxiety in respect to all the docks, and they have been taking care of them as fully as possible. It would be necessary to be more careful in docking a ship in a wooden dock than in a stone dock, and a ship can be handled in a much more rough and ready manner in the latter than in the former, without danger of injuring the dock.

In handling material in a wooden dock care must be taken not to destroy the altars, a necessity which does not obtain, to as great an extent, in a stone dock. Less care is required in wedging against a stone dock than against a wooden one, though the speaker does not believe that any wooden docks have been injured by docking a ship, so far as he knows, nor have any ships been injured on account of docking in a wooden dock. But as the docks decay, of course injury may happen.

The speaker thinks that from the nature of the two materials, stone and wood, equally good construction cannot be obtained, and it is hardly possible to make comparison as to strength or durability. Sufficient timber could not be put into a wooden dock to afford a proper comparison with a structure of stone and concrete. In the speaker's experience, the construction of some wooden docks has been rather on the slimsy order of mechanism and construction, though he thinks the Anglo-American Company worked honestly and did the best they could with their wooden docks. Undoubtedly, the builders of wooden docks did the best they knew how, and exercised care, except perhaps in the case of a certain timber dock recently constructed.

The speaker can furnish no data as to the relative cost of good concrete or good wooden docks. It is only when a contract is made with responsible people that the relative cost of the two classes of docks can be ascertained. Wooden docks can be built for $1 000 per lineal foot, and stone and concrete docks can be built for $1 500 per lineal foot.

In the early days the speaker was in favor of wooden docks. Now, he is in favor of stone, or, rather, concrete and stone. He believes that in the future dry docks should be constructed of concrete, the best that can be made, faced or lined with stone, particularly in the wearing parts of the dock. There are parts of the dock, the wings and the angles, for example, that need not necessarily be of stone, as concrete will answer the same purpose.

Mr. Asserson.

In the Norfolk dock the upper part was recently rebuilt entirely of concrete, which made a very good job. The only reason for putting a wooden lining on the front of the concrete altars was to prevent the abrasion of the green concrete. The upper part of the Norfolk dock was repaired while the dock was in daily use, and the speaker thought it was necessary to do something to protect the concrete during construction. If he had had the money, he would have put in stone instead of wood, but the latter was put in in such a manner that when the planks split or became decayed they could be taken off and replaced by others.

The speaker can see no reason why dry docks constructed wholly of concrete, and without stone lining, should not be satisfactory, if the corners of the altars are rounded off. He would prefer, however, to make assurance doubly sure, and use stone on the wearing parts, for the reason that there might be carelessness in shoring up ships. There is a probability that the corners of the concrete altars might be broken off, particularly where they are so shallow—only 8 ins. high—as in these structures. With a larger body of concrete, say 12 or 18 ins. high, the speaker would not hesitate to allow the use of altars wholly of concrete, because the corners of the altars themselves need not be touched. The corners of the concrete are sometimes broken off by sliding timbers and heavy material over them and down into the dock.

In reference to the lining of the timber docks with creosoted plank, it is a fact that the teredo will not attack it for several years. The teredo will not pass across a water-bearing seam. But creosote will protect only for a time. The speaker's experience has been that where creosoting has been throughly well done, a good, heavy, dead-oil creosote injected into the timber at a pressure of 100 or 120 lbs. per square inch, it will protect timber for from 10 to 15 years; after that the creosote is not so effectual. Unfortunately, the creosote used in the altars at the Norfolk dock was not the best kind. The speaker thinks it was what is called "pine oil," and he believes it helped to rot the timber more than to preserve it. However, the top of the dock at Norfolk is badly decayed for a distance of 8 to 9 ft. down from the coping.

The timber dry docks with which the speaker has had experience have all been constructed in about the same manner. There is a difference in the shape, but they have been practically earth excavations, having the sides lined with timber, the slope of the ground being such that it would stand by itself, except that at the Brooklyn yard the ground does not stand anywhere, but moves and sinks as soon as the mica sand has been reached.

Most of the Government's timber docks are good, but the speaker thinks there is room for improvement as to their strength, especially as regards the sides; the bottoms of the docks will no doubt last many years.

Mr. Asserson. There is no question in the speaker's mind but that in these days the Government should build for the future. Wooden docks were adopted by the naval service, not because of their durability, but because they could be built quickly and cheaply. Our need of docks was such that we could not wait for stone docks to be constructed. For 20 years Congress was utterly unwilling to appropriate large sums for dry docks, for repairs in the navy yards, or for building up the shops that were decaying.

Now that Congress and the country are willing to appropriate money, it is time to abandon that policy. Instead of building wooden docks, that serve at most only for 20 or 25 years, structures that will serve generations to come should be built.

The estimated cost of a concrete dock, 750 ft. long and lined with masonry, for Norfolk, Va., was $1 100 000. For a timber dock of the same size, and built as strong as possible, the estimate was $700 000. The estimate was for a concrete dock lined with stone 3 ft. thick, with stone abutments, and for a steel caisson, including pumping machinery. The stone lining was to have been set in the main dock chamber, far enough into the concrete to be firm and to form a tread or step, and with a very open slope—30 to 33°—so as to admit light and air, the same as for a wooden dock. There is no doubt that the shape of wooden docks is very good, because light and air are so readily admitted, and because access is so easy.

If first-class timber, or heart timber only, was put into a dock, and in a first-class manner, the dock would last 5 to 8 years longer than ordinarily, but all heart timber can hardly be obtained in such large quantities.

The wooden docks have served well to the present time, and the speaker thinks it is best to praise the bridge that carries us safely over. Nevertheless, he is of the opinion that we should now adopt material of a more durable character for structures that we expect to last and use for hundreds of years.

Mr. Tait. John G. Tait, Assoc. M. Am. Soc. C. E.—No one makes any attempt to deny the fact that a masonry dry dock is much more desirable as a government structure than a timber dry dock. Therefore the item of first cost must be the argument that influences the Congressional Committee on naval affairs when they decide to build timber instead of stone docks. As engineers, citizens and taxpayers, we are all interested in having the Government make the best investment, and build masonry docks, and to that end, until the Naval Committee are educated up to the advantages of stone docks, and while the engineers are still in competition with men advocating timber docks, any unnecessary cost should be omitted from the masonry docks.

At the site of the Boston dock the cost of concrete per cubic foot is less than half that of timber. This dock contains a great deal of gran-

ite, the stones are very deep, and, according to the plans and specifi- Mr. Tait.
cations the radial lines have to be preserved for the full depth of the joint, which makes very expensive cutting. Then, in addition to that, the same courses are continued from the main part of the dock out into the jaws, so that each stone constantly increases in width as well as being a voussoir.

These requirements probably make the cost of the dock about $200 000 more than it would have been if built entirely of concrete, or of concrete with granite quoins. If the dock had been designed with a smaller lining, and if there had been a different arrangement of the stones in the jaws, the bid would have been very close to that of a timber dock.

JOHN KENNEDY, M. Am. Soc. C. E.—Having recently examined all Mr. Kennedy.
the large dry docks on the Atlantic coast from Halifax to Norfolk, and on the St. Lawrence and Great Lakes, from Levis to Lorain, mainly with reference to the choice of material for the construction of a dry dock of large size at Montreal, the speaker will state some of the results of his observations.

The wooden dry docks of both public and private ownership show general weakness of construction, and a rather rapid decay. The decay is, of course, most rapid in the parts above water level, and it appears to reach farther down and be more rapid in docks in fresh water than in salt. For example, beginning at the North, the wooden dry dock at Portland, Me., built in 1869, has had no recent repairs, and its altars and pile heads above low water are entirely rotten.

At Boston the altars and other lining of the three wooden docks built in 1854 to 1864 have been practically all rebuilt inside, and have required frequent repairs. The largest one is leaking beneath the entrance. The two private docks at Erie Basin, Brooklyn, built in 1866, were all relined in 1881, and concrete substituted for wood in the upper five altars. The wooden dock at Cramps', Philadelphia, built in 1876, has largely failed. The altars and the slopes near the abutments are bulged in badly at places; the altars, from weakness and decay, have been stove in at places, revealing cavities behind, from which the earth slopes have been washed down. The League Island dock of the United States Navy, built in 1891, is notably weak, and has bulged in badly. While occupied by the United States cruiser *Columbia*, a month ago, the abutments were shored up to prevent collapse. There are serious leaks of water and mud in the bottom and lower altars of one side, and the earth slopes behind the altars have run down in several places, leaving large cavities. Repairs, estimated to cost about $64 000, are about to be undertaken. The Norfolk Navy Yard dock, built in 1889, is now having the upper eleven altars renewed in concrete because of the decay, not only of the altars, but of the braces and pile heads which carry them. The

Mr. Kennedy. wooden dry docks visited on the Lakes are in much the same general condition.

The speaker found that all engineers of the United States Navy Department who have had experience in the working and maintenance of wooden dry docks, and with whom the question was discussed, were understood to be quite against wooden dry docks, at least for naval use, because of their weakness and short life. On the other hand, private owners of wooden docks are in favor of wood for their own service, and in adding a new dock they build either wholly or mainly of wood. The obvious difference of conditions will account for much of the difference of opinion. In Government docks a small saving in first cost is of minor importance, but weakness and frequent need of repairs are well nigh intolerable. On the other hand, in docks of private ownership, saving in first cost is usually of serious importance, while the cost of maintenance repairs is met by the earnings of the dock, and is less felt. It is to be noted, however, in the more recent private docks built by owners of experience, that there is a desire for something more permanent than wood. A notable instance is that of the large dock now being built by the Newport News Shipbuilding & Dry Dock Company, in which the abutments and wings are to be of concrete. In the Erie Basin docks, Brooklyn, the wooden altars above the water line have been replaced by concrete. It may be noted, too, that all the recent wooden docks, of which the speaker has knowledge, and the more ancient ones, also, have not been built under any patents, nor by contractors holding patents; or, in other words, while the building of wooden and partly wooden docks is still continued by private firms, the patented kind seems to have passed out of favor.

In Canada there are no wooden dry docks of importance. All the Government docks, three in number, are of stone and concrete. One of these, on the St. Lawrence, at Levis, Que., was finished in 1886; one on Lake Ontario, at Kingston, Ont., was opened in 1892, and the third, on the Pacific Coast, at Esquimault, B. C., was finished in 1887. All are of excellent limestone ashlar, with concrete backing and inverts, and none have had any repairs of more importance than pointing, except in the caissons and other accessories.

Much reduction in the cost of stone dry docks has been made, by the substitution of concrete for part of the masonry, and it is proper to consider how much farther such substitution may be wisely carried, in cases where economy is of prime importance. The masonry of a stone and concrete dock is, of course, that in the linings, altars, copings, inverts, entrance and other parts where it is requisite that it be of cut ashlar, of expensive kind; and the further elimination of the whole or any considerable part of such costly work, rapidly reduces the average of the whole. There can be no doubt as to the strength

of solid concrete dock walls being fully as great as that of walls of concrete and ashlar combined, in which the bulk of the wall is of concrete and the ashlar is a mere facing. The real question is as to the wearing and weathering qualities of the inner surface and of the altars, if of concrete only. Examples of concrete of ample durability as well as of strength, can be pointed out, which show that good concrete will fulfill all needed requirements, even in northern climates. The making of Portland cement has also been so wonderfully improved, and the testing of it has become so simple and reliable, that there is now no doubt as to the obtaining of cement of an excellence and uniformity which was unattainable a few years ago. Mr. Kennedy.

In England and throughout Europe, dry docks, wet docks, arched bridges and other important structures, made almost wholly of concrete, are becoming common. At Erie Basin, Brooklyn, there is a minor, but, so far as it goes, a very satisfactory example of the durability of concrete, in the altars and copings of dry docks in this climate. The upper five altars of the two wooden docks were, as already stated, taken out and rebuilt wholly of concrete, and their condition now, after eighteen years of constant use, is excellent.

The Halifax, Nova Scotia, dry dock, of 580 ft. length, built in 1886 to 1889, is all of concrete, except the meeting faces of the caisson gate, the coping courses of the altars and main walls, and the stairways and material slides, which are of granite, and the linings of the culverts, which are of brick. The inverts of the entrance are all of concrete except the meeting faces. The example, though not wholly satisfactory as to the durability of the concrete, is a very instructive one. Considerable areas of the inner surface of the body walls have scaled off, and have been relined with blue bricks, but the failure is readily accounted for by much of the walls having been built under most unfavorable conditions as to storms and frost, and storage of cement. On the other hand, adjoining parts of the main walls, and all of the inverts and walls of the entrance, are standing excellently well. Altogether, the dock shows that really good concrete is a proper material for dock walls, even in a rigorous climate.

A notable example of a very large dry dock, with walls almost wholly of concrete, is the new No. 3 Graving Dock at Glasgow, designed by Mr. Deas, Engineer of the Clyde Trust. It is 880 ft. long, 115 ft. wide at top, 32 ft. depth from floor to coping, and is divided into two sections by gates. The side walls are entirely of concrete, except a thin backing of brickwork placed against the sheet piling which surrounds the walls. A peculiar and what appears to be an excellent method of construction of the walls is that their whole inner faces are made, not out of concrete built *in situ*, but of granolithic concrete blocks, separately molded and hardened, and then built into place as ashlar. The altars, up to about half the height of the walls,

Mr. Kennedy.

are 3 ft. 6 ins. high and 20 ins. tread, and from that to the top of the walls are 18 ins. high by 14 ins. tread, the granolithic blocks being molded to the height of the altars in each case, so that a course of blocks builds an altar. The coping of the top of the walls is also of granolithic blocks. The head of the dock is semi-circular, without altars, and is lined with concrete blocks which are of heights corresponding to the adjoining altar courses, but these blocks, not having exposed tops, are made of ordinary concrete with only a granolithic facing 3 ins. in thickness.

An older example of a concrete dry dock in Britain is that at Newport, completed in 1890, of 350 ft. length by 30½ ft. depth, and, although built on a very difficult site, its cost was only $121 500.

An important example of concrete construction, for purposes analogous to those of dry docks and exposed to somewhat severe weather conditions, is that of the locks of the Trent Canal in Ontario, now being built by the Government of Canada.

The advantages in strength, durability and economy of maintenance of dry docks of stone, and of stone and concrete, over those of wood, have been ably and fully shown in the discussions presented; and, for the reasons just outlined, the speaker adds a plea for dry docks built almost wholly of concrete, as possessing much of the advantages of both.

Mr. Ritchie.

JAMES RITCHIE, M. Am. Soc. C. E.—The speaker built a dry dock recently, but it was of wood, as is customary on the lakes. He suggested that the abutments be built of stone, but the cost of such work was the objection made by the owners, and only a small amount of concrete was used. It is a large dock, 560 ft. long and 100 ft. wide, but, as has been stated, it has the one advantage of a wooden dry dock—it is shallow, only 17 ft. on the sills; consequently it is considered perfectly safe, and has so far been extremely satisfactory. For a dock of great depth, or for docks where the first cost is not the essential matter, stone, or concrete lined with stone, is, in the speaker's opinion, the only proper material to use.

Mr. Collingwood.

FRANCIS COLLINGWOOD, M. Am. Soc. C. E. (by letter).—The original cost of wood construction is, of course, less than stone. The repairs and renewals are a matter of experience, and it is claimed by those having long experience with wooden docks, that the interest on the difference in cost between them and stone will more than pay for keeping the former in first-class condition. The reduced rate of interest now prevailing, however, may cause a change in this regard.

As to convenience, etc., those who are constantly using docks are most competent to speak. Such matters as length of shores and convenience in adjusting them, convenience in handling material and, in movements of men, slipperiness and cold to the feet, contents to be pumped out, and other practical matters have to be considered.

As the writer's connection has been with construction only, others can speak with more authority. Mr. Collingwood.

It is to be regretted that the experience in building some of the recent timber dry docks has in a measure thrown discredit upon them. Errors in judgment in engineering matters are not justly chargeable against a system of construction which has certainly proved successful in many existing examples.

Charles H. Haswell, M. Am. Soc. C. E. (by letter).—The writer thinks it is to be regretted that the subject under discussion is confined to the relative fitness of the material of a dry dock, inasmuch as piers and bulkheads, erroneously termed docks, are of more general requirements and consequent utility than docks either wet or dry. Custom and some lexicographers term the enclosed space between two piers a dock, whereas it is strictly a basin, although colloquially, whether the structure is a single pier or a bulkhead, it is termed a dock. Mr. Haswell.

If the subject had included piers and bulkheads, whether of masonry, crib-work or rip-rap, the writer would avail himself of the opportunity to refer to their design and construction, on the basis of his experience and observation; but as the subject is confined to the question of the relative merits of stone and wood for the construction of dry docks, it is a restricted one, as it embraces only the relative cost and endurance of such structures.

If the period of construction and the first cost are controlling elements, wood alone can be entertained, as it involves less cost of material and labor in construction, not only in its walls, but in its foundation. Its defective elements are decay and damage by fire, involving the probable damage or destruction of a vessel under repairs.

If, on the other hand, the period of its construction and its first cost are not restricting elements, and a permanent structure is required, stone or concrete is not only indispensable, but, in the matter of ultimate cost, it is questionable if frequent repairs, and the periodic renewal of a wooden structure, would not involve a greater cost than stone.

Francis T. Bowles, Esq.* (by letter).—After thirteen consecutive years' experience in the active use of wooden and stone dry docks at the Navy Yards at Norfolk, Va., and New York, and after having had sole charge of the repairs to the wooden dry dock, known as No. 3, at the latter place, in which it was necessary to reconstruct the whole entrance, the writer wishes to express an unqualified preference for a masonry or concrete dry dock over wooden construction. In testifying before a Congressional Committee about a year ago, the writer expressed the following opinion: Mr. Bowles.

* Naval Constructor, U. S. N.

Mr. Bowles. "I believe that a wooden dry dock bears the same relation to a stone dry dock or concrete dock, that a wooden trestle or a wooden bridge bears to a steel bridge or stone culvert. I consider it a temporary structure."

This opinion appears to present the case fairly.

The wooden dry docks built 10 or 12 years ago for the Navy have served their purpose well; they were cheap, and admitted of easy and rapid construction by unlearned persons. They were built at a time when little attention was given to the subject of dock construction in this country, and when materials for stone and concrete work were very expensive. Moreover, these docks supplied an immediate and pressing need which at the time could not have been otherwise met. It is now known that the deterioration which they have developed and the essentially temporary character of the work and material of their construction, show that their continued use involves danger and great expense for repairs.

Consideration of the most elementary principles of engineering leads to preference for permanent rather than temporary construction where cost is not prohibitive, and at the present time the cost of concrete and stone construction is sufficiently low to require of the Government the most durable, safe and best construction of its dry docks.

This opinion is not derived from the recent bids on the new dock at Boston, which, to the writer, appear to be inconclusive. He is strongly opposed to the letting of contracts for the building of a whole dry dock to one person or corporation, on the ground that the nature of the undertaking involves too many undetermined elements to enable any person to make a bid which will enable him to do the best work throughout. The work should be under the sole charge of a competent engineer, who should control absolutely the conduct of the work, and let contracts from time to time for definite portions, on which clear and definite specifications can be made. That is good business, and in the end will produce the best results. This digression from the main subject will possibly be pardoned, because, in advocating stone or concrete docks, it is the writer's opinion that the engineering problems are great, and must be treated by modern methods.

Mr. Boller. Alfred P. Boller, M. Am. Soc. C. E. (by letter).—It is astonishing that a controversy regarding the relative merits of stone and wooden dry docks should be given serious consideration by a great government like that of the United States. That such a controversy does exist is a sad commentary on the intelligence of the source of ultimate power, the National Congress. There are some questions that have only one side, and this is one of them. It is so thoroughly one-sided that only the crass selfishness of interested parties, operating in some mysterious way upon Congress, forces a consideration of the subject at all. Is there another civilized government under the sun that would thrust aside the united demands, yes, pleadings, of its technical ex-

perts to do the right thing, simply for the sake of maintaining a special private interest, whose business is building wooden docks? Mr. Boller.

Stone or wooden docks! As well ask our railways to build wood instead of stone or steel bridges. The only so-called arguments advanced for wooden dry docks are those of economy and expeditious construction. Economy of first cost is one thing, and economy of endurance and maintenance is another. Rapidity of construction is a matter of little importance for a Government work, compared to solidity and endurance, but modern methods of executing work permit of such rapidity of performance that almost any reasonable expectation of time can be met. What if it does take twice as much time to build a stone dock as a wooden one; when the work is done it is done for all time practically, and it can be operated with absolute confidence and freedom from the anxiety that always attends a wooden dock. Absolute safety and economy of endurance are the cardinal requirements for any dry dock suitable for the purposes of a modern navy, with its deep-draught battleships, which can only be met by using imperishable material. To build such docks of wood, that commence to depreciate from the day of their completion, deteriorating increasingly with the years, until absolutely worthless, is serious folly that is belittling to our naval administration, a waste of the public funds, and a source of ridicule with competitive nations. A wooden dry dock is all well enough for temporary purposes, or to meet the necessities of an impoverished government, or the financial limitations of a private party. The United States is in no such impecunious position, and is expanding into a naval power, the limits of which no man can foresee. This country is destined to a restoration of a merchant marine that will recreate the long-lost glory of the seas, now only a reminiscence, and on a scale commensurate with the activities of a hundred million of people, backed with unlimited resources. For the Government of such a nation to even talk of building wooden dry docks, is preposterous. While it is gratifying to know that the comparative cost between the two systems of docks shows, perhaps, 25% difference (according to recent tenders), the question of first cost, to the writer's mind, cuts no figure at all. The United States cannot afford to build wooden docks, even on a simple financial comparison, as the statistics so elaborately discussed by the naval speakers, demonstrate beyond cavil. Whether stone dry docks cost much or little, whether they take twice as long to build as wooden ones or not, there is but one thing for the United States Government to do, and that is to build stone docks, even at the risk of hurting the feelings of existing wooden dry dock companies.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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PAPERS AND DISCUSSIONS.

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DISCUSSION
STREET GRADES AND CROSS-SECTIONS IN ASPHALT AND CEMENT.*

By Messrs. P. W. HENRY, JAMES OWEN, D. ULRICH, EDWARD P. NORTH, G. W. TILLSON and A. W. HANKINSON.

Mr. Henry. PHILIP W. HENRY, Assoc. M. Am. Soc. C. E. (by letter).—Although the engineering problem to which this paper calls attention is not a difficult one, yet it is one which is met continually by city engineers and is seldom solved satisfactorily. This, however, is not due to its intricacies, from an engineering standpoint, but because the solution is complicated by politics and the influence of property holders. It is an exceedingly difficult task for an engineer to improve a street, where the conditions are similar to those mentioned in the paper, without arousing the determined opposition of those most nearly interested. For the sake of saving a few dollars the average property holder is willing to allow any kind of a grade to be established; and he objects most strenuously to any plan which will cause him to raise or lower his sidewalks or alter in any way his buildings, no matter how old or dilapidated they may be, or no matter how much the improvement may benefit his property eventually. It follows, therefore, that a city engineer is seldom able to carry out his own convictions in establishing a street grade, unless he is willing to make a determined fight, or use all the persuasive powers at his command. In Wabash, it appears that either

* This discussion (of the Paper by Robert P. Woods, Jun. Am. Soc. C. E., printed in *Proceedings* for February, 1899), is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication. *Proceedings*, p. 71.)

Communications on this subject received prior to April 22d, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

the engineer's persuasive powers are highly developed, or else the property holders there are unusually enlightened, for the owners of the buildings affected, in several cases, waived all claims for damages occasioned by the change of grade. Mr. Henry.

It also appears that the author did not attempt too much in improving the grade of the street. The change from the old condition was radical, but at the same time it was no greater than the circumstances warranted. The difference in level between the two sidewalks was met in a somewhat novel manner, by putting in a step on the upper side of the street. It is seldom that this construction is used, and it could doubtless be introduced in other cities to advantage. Its chief disadvantage is that it lessens the width of the roadway, and on a narrow street this would, of course, be objectionable. The ample width of the Wabash roadways allowed this construction, with excellent results. The illustrations on Plate V show what a decided change was made in the appearance of the street by the improvements described in the paper. To one who saw the street before and after the improvement, the change is even more marked than that shown in the photographs.

To return to the details brought out in the paper: It does not appear why the crown was placed in the quarter in Sections 1 and 2, and not in Sections 7 and 8. It is generally objectionable to have all the water drain to one side of the street, as shown in the two latter sections. It would also seem that in these two sections each step could have been made 0.85 ft., the greatest limit, and thus have reduced considerably the slant of the street from one gutter to the other.

Although the property holder has undoubtedly been put to considerable expense by the radical change in sidewalk grade, yet it is obvious that any less radical treatment would have given a patch-work effect to the street, which would have been a continual source of annoyance. There are too many streets in prominent cities which are anything but creditable to the engineers who have established the grades. This is generally due to poor engineering talent (if any engineer was employed at all), in laying out the street at the start, for many streets were originally country roads which no one foresaw would one day require paving. By the time the street had arrived at the dignity of a pavement, buildings had been erected at different grades on each side, so that the engineer who had the improvement in charge was at his wits' end to know the best way to deal with the problem. In solving this problem no general rule can be laid down, but each case must be treated by itself, taking into account the many different questions which are involved. Among these questions are width of street, value of real estate and improvements affected, amount and kind of traffic, probable increase in real estate values and probable increase in traffic. These questions are being met every day by municipal engineers all

Mr. Henry. over the country, and are being solved in a better and more satisfactory manner than ever before. There is no doubt that the city engineer of to-day is the superior of his predecessor of twenty years ago, and brings greater talent and broader knowledge to bear on the questions which are continually before him. It is to be hoped that the paper will bring out discussions from engineers who, from practical experience, are able to give much interesting and profitable information to the profession.

Mr. Owen. JAMES OWEN, M. Am. Soc. C. E.—In regard to the question of crown, the speaker's ideas are somewhat decided. It is his desire to impress upon everybody the fact that the roadway itself is made primarily for wagon travel, and that, *per se*, a level cross-section is much more advantageous for such travel than any other cross-section. A departure therefrom can only be tolerated because there are overcoming disadvantages against it. The original country road was piled up probably 2 ft. in the center so that the water could be shed to the gutters. As the character of highways has been improved, the crown has been gradually decreased, until, with the improved pavements of to-day, it seems to the speaker that the crown is scarcely a matter of consideration, and in his practice, in both asphalt and brick pavements, he has reduced it to about 3 ins. in, say, 40 ft.

The only advantage of the crown is to shed the water to the sides in order to avoid the wash of the perishable material of the roadway and the damage ensuing therefrom. For a permanent, smooth roadway, either of granite blocks, brick or asphalt, no argument can be advanced in favor of any crown at all, except such as is needed to shed the water, at the proper point, away from the street entirely. In fact, the question of using the roadway itself, from curb to curb, as a gutter, is now quite often considered; and the question of economy in constructing storm sewers is being considered in that light. For instance, on a side hill, with streets at right angles to the slope, it is economical to use the streets themselves as gutters and discharge the water into the storm sewer at each cross street, the latter being provided with a crown in order to divert the water into the sewer. The speaker thinks that practice is going to be more in vogue than it has been.

In considering the elimination of crown as much as possible from street construction, it will be seen that these difficulties will disappear entirely. The speaker agrees with Mr. Tillson that he could have solved the problem without the crown, and could also have avoided the steps in the gutter line.

The relation between the roadway and the depth of the gutters is purely arbitrary. In the business part of Yonkers, two or three years ago, the speaker noticed that, in the improved streets, the sidewalks were from 18 ins. to 2 ft. above the roadway. That, of course, had come about from the fact that the original grade was being altered,

and, in permanent improvements, some kind of a grade had to be es- Mr. Owen.
tablished, without relation to the sidewalks. The result was that the sidewalk was adapted to the improvement of the house, and it would have been detrimental to the property to cut the sidewalks down; that the sidewalk, being 2 ft. above the street, offered no practical disadvantage, and that wagon travel could be more evenly handled with a sidewalk 18 ins. high than with one only 8 ins. high. A person can step out from a carriage more easily, and can more readily transfer freight to or from a wagon when the sidewalk is 18 ins. high than when its height is only 8 ins., and in this instance the foot travel on the spaces between the cross streets is very small. By making proper arrangements at the street intersections for the accommodation of foot travel passing from the sidewalk to the street, no practical or permanent disadvantage will result in having the sidewalk higher than the stereotyped 8 ins. Furthermore, the saving is a disadvantage, in any way it can be considered, both by stepping from a carriage a larger distance from the sidewalk and also in carrying freight over that space.

The question of the relation of the sidewalk on one side of a street to that on the other is continually arising. The speaker has constructed roadways in which there was a difference of 4 ft. in the elevation of the sidewalks. Of course, that would not be considered an ideal construction, but it had to be used. In such cases a standard curb was adopted, and the crown of the roadway was placed about 6 or 8 ft. from the center toward the higher side.

D. Ulrich, Assoc. M. Am. Soc. C. E.—In constructing asphalt Mr. Ulrich.
pavements the speaker's experience has been limited, and has generally been in localities where no curb lines have been established. For that reason he has had no trouble in making the crown suit the width of the road, and has in nearly all cases made it about 4 ins. for a 60-ft. roadway. In constructing macadam roads, however, he has ordinarily made it somewhat higher. For a 28-ft. road the crown was generally made 4 ins.; for a 30-ft. road, 6 ins.; and for a 40-ft. road, 8 ins.

Edward P. North, M. Am. Soc. C. E.—In what is now the Mr. North.
Borough of Manhattan, in the City of New York, under the old charter, the Water Purveyor had charge of all new paving, repaving and repairs to pavements. New pavements, and all grading, regulating, curbing and flagging, were paid for by assessments on the property benefited. All repaving and repairs to the pavement, except those due to openings in the streets made by plumbers or companies having pipes or conduits under the pavements, were made by the city and paid for out of appropriations.

City ordinances require sidewalks to have a rise of 1 in. per 5 ft, from the curb. The lower part of the city is mostly without official grades, but commencing about Houston Street grades have been established. It has not, however, been established whether such grades

Mr. North. refer to the curb stones or the middle of the streets. This uncertainty is, at times, a matter of convenience to the engineer who wishes to improve the profile of a street that is to be repaved, as it allows a comfortable working latitude.

There is a noticeable exception to this statement, viz., the streets joining the Riverside Drive. Here, the grades were established by the Park Commissioners for the curbs, both on the drive and on the streets back to the house lines. Unfortunately, they were substantially level, although the same authorities had established quite steep grades on the streets (in one instance, though not adjoining the Riverside Drive, a grade of 18%, which is thought to be the maximum). As any departure, in either paving or curb setting, from the established grade, is liable to result in vacating the assessment, these level areas of 18 to 20 ft., joining, say, a 10% rise, have to be retained, and are objectionable.

There is also a troublesome practice by those who are called "skin builders," who, contracting to build a house with certain dimensions between the floors, avoid the excavation of 2 and sometimes 3 ft. of rock, by establishing a curb grade suited to their work and also by increasing materially the slope of the sidewalk. This swindle is generally undetected, particularly on unpaved streets, until the house is paid for, and, in some instances, causes considerable trouble and dissatisfaction to the property owner, as the curb is generally reset on the proper grade when the street is paved, which either necessitates relaying the sidewalk in front of the property, or an approximation to the practice described by the author.

Except as in the instances cited, the engineer in charge of the streets in the City of New York probably has as little trouble with grades as in any other city or town. No ordinance governs the depths of gutters, but they are called for in all streets; nor is any crown specified in the city ordinances.

On taking charge of the paving in what is now the Borough of Manhattan, with the office of Water Purveyor, it was decided to make the crown of all asphalted streets, with curbs at equal elevations, $\frac{1}{100}$ of the distance between curbs, and the normal depths of gutters between $3\frac{1}{2}$ and $4\frac{1}{2}$ ins. The crown adopted, combined with the low curbs, gives a street of apparent civilization and luxury; the high crown being a survival from the pathmasters' "turnpiking" with heavy soils; and the deep gutters antedate sewers. Some of the asphalting companies objected to these unprecedentedly flat transverse profiles, as the cost of thorough maintenance is, doubtless, greater than where water will run off quicker. It was with some satisfaction that notice was taken that the French engineers had at last adopted the same rise, viz., $\frac{1}{100}$ for the wheelway of the Alexander III Bridge, drawings of which were presented at the meeting of the Society on February 15th, 1899.

Mr. North.

No complaint has ever been made, to the speaker's knowledge, of cellars flooding through insufficient capacity of $3\frac{1}{2}$ in. gutters; so it apparently may be assumed that on asphalted streets with 4-in. gutters and sidewalks of one-half the width of the wheelway, rising at the rate above mentioned, no danger to cellars is to be apprehended if the sewers have sufficient capacity to take off the water.

Objections are frequently made to longitudinal grades on asphalted pavements, futile efforts having been maintained for years by one of the city departments to confine asphalt to grades of $2\frac{1}{2}\%$ or less. Small attention, however, is generally given to transverse grades, although every one knows that while on asphalt a horse may stumble in the line of draft, he almost invariably falls at right angles to it. As a matter of fact, it is difficult to get a horse that knows anything about asphalt to travel anywhere except in the center of a high-crowned street in slippery weather. This obvious danger from excessive transverse grades led to the suppression of the crown in streets having a transverse fall of much over 3 per cent.

The first asphalted street paved without a crown was Eightieth Street between Fourth and Madison Avenues. On Fourth Avenue the difference in elevation between curbs was 1.3 ft.; by making a 7-in. curb on the north and 4-in. curb on the south side, the fall was reduced to 1.05 ft., or a grade of $3\frac{1}{2}$ per cent. The foundation was the old trap-block pavement relaid. The transverse profile at Fourth Avenue was a right line; at Madison Avenue it had a rise of $\frac{1}{100}$, and the south curb was somewhat lower than the north curb. The grades were "boned in" for 405 ft. between the two end-profiles on the center, quarter and curb lines. No settlement nor distortion has been observed.

The above-mentioned practice was often modified, where the relative heights of the curbs would admit, by reducing the length of the warped surface to 150 or 200 ft. In other instances it was thought advisable to dispense with a crown for one or more blocks. In such cases there was no gutter on the upper side. No instances have come to the speaker's knowledge where sags or depressions in the foundations have developed, either with a relaid stone pavement or with a concrete foundation. In some cases, however, a crown of about 1 in. was worked in, consequent to the turnpike idea, as aforesaid.

Experience in the City of New York is decidedly against concrete curbs. While a good curb can be made of concrete, with proper materials and care, poor work cannot be prevented, and many, if not most, of the concrete curbs laid have been scamp affairs, with concrete made of cinders and plastered with a thin coating of poor cement. The wheel of a grocer's delivery wagon would cut this thin coating off, and backing a coal cart against the curb would break it. The poorest curbs were put down in the tenement house districts where the surroundings are squalid enough without the additional aid afforded by

Mr. North. broken curbs. Fortunately, there was a city ordinance requiring stone curbs. Under this ordinance, samples of the curbs were taken to court and their construction, for a time, at least, stopped.

Mr. Owen's suggestion, that the gutter should be in the middle of the street, is worth consideration, particularly for asphalted streets. Rain would flush the pavement more quickly, and as the transverse profile would be two right lines joined by a concave curve, instead of a convex curve increasing in rate toward the side gutters, as at present, the whole carriage-way on level streets could be made equally available for traffic, as it would not be necessary to have the transverse grade as high as 1 per cent. A single gutter in the center of the street is found in many Spanish towns, and was considered for some of the narrow streets in the lower part of the City of New York, but the ordinance calling for gutters and curbs prevented its use.

Mr. Tillson. G. W. Tillson, M. Am. Soc. C. E.—The speaker has been much interested in this paper, as it discusses a question which he has been studying for some years. The author has been confronted with the problem that always presents itself to every engineer who attempts to pave a street where the grades have been established by one who never expected the street to be paved, or had no idea of the proper cross-section of a paved roadway; or where a side-hill street has been built up, with no consideration for the future improvement of the street, but only according to the whims of the individual property owners.

When such conditions exist, and the entire width of the street is to be improved, the difference in elevation of the curbs must first be reduced to a minimum, by giving the sidewalks on the high side the maximum, and on the low side the minimum, allowable slope. In the same way, by applying the maximum gutter to the high side, and the minimum to the low side, the difference in elevation of the gutters is made as little as possible.

The best cross-section for the roadway itself is now to be considered. Some engineers would make this a straight line from gutter to gutter. The speaker does not think this is right. Any little variation from a true plane, caused either by imperfect work or the action of travel, will show much more plainly than when the surface is slightly curved. It is also better to prevent all the water from running transversely across the street. By locating the crown of the street on the quarter, instead of at the center, and by giving a slight rise from the higher gutter to the crown, these two difficulties are overcome, the difference in elevation is but slightly changed, and the distance in which to overcome it is made considerable.

The author has adopted 0.33 ft. for a minimum and 0.85 ft. for a maximum gutter. These figures agree very closely with the speaker's ideas; but a gutter involving a step of 10 ins. should be used only in very extreme cases, and when to this is added another step of 11 ins.,

and the two are separated by a tread of 12 ins., the construction is bad, and should be avoided, unless it can be shown to be absolutely the best under the existing conditions. A double curb, as constructed, is expensive, unsightly and dangerous. Mr. Tillson.

In Fig. 4 the speaker has taken five of the worst sections shown in the paper and has replatted them, and on the same base has platted the sections he would have used. The author's section is shown by a full line and the speaker's by a dotted line, the latter result being obtained by using sidewalk slopes and gutters within the limits recommended by the author.

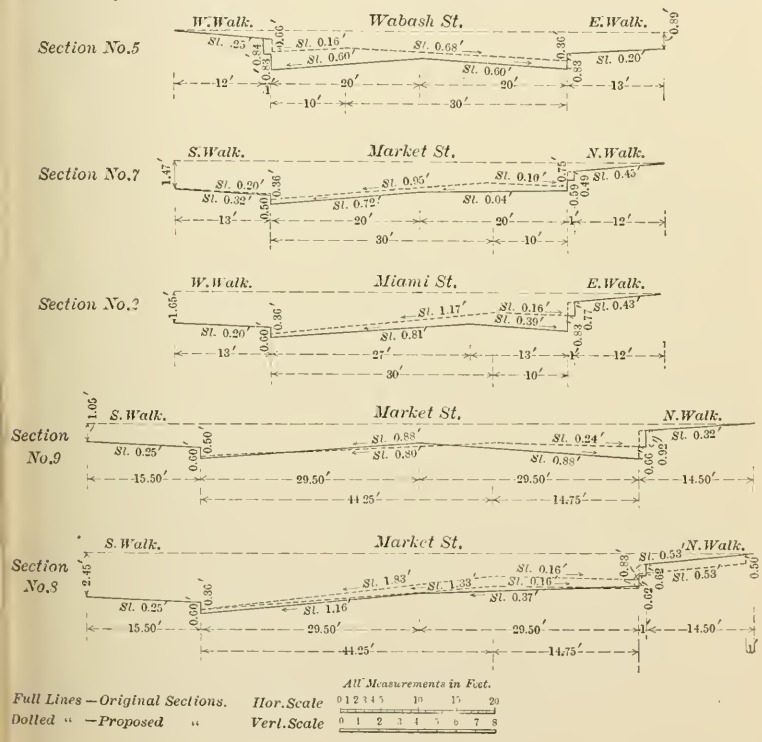

FIG. 4.

In Section No. 5 it will be seen that by making gutters 0.66 ft. and 0.36 ft. on the high and low sides, respectively, making the crown on the quarter, and allowing a rise of 0.16 ft. from the gutter to the quarter, a section can be obtained which in no place has a greater slope than the author's, which has a step of 0.66 ft. instead of 0.82 ft. and which does away entirely with the double curb. On the street, the fact that the two sides are unbalanced would hardly be noticed, except

Mr. Tillson. by a trained eye, and for practical use this section is as good as the other.

In Section No. 7 the slope of the sidewalk on the low side is reduced to the minimum and the depth of the gutter made 0.36 ft. On the other side the maximum gutter is taken, and the rise to the quarter is the same as used in Section No. 5. The resulting section has no slope in excess of the author's, does away with the double curb, and has a curved surface where that shown by the author is practically level, there being but 0.04 ft. fall in 20 ft.

In Section No. 2 the gutter on the low side is made 0.36 ft.; on the high side it is placed at the top of the author's real curb, and the section is figured as in No. 7. This slope is slightly in excess of that used by the author, but not to such an extent as to be detrimental, while on the other hand it does away with the necessity of the double curb.

Section No. 9 is on a street 90 ft. wide. The low gutter is made 0.5 ft. deep, and on the high side it is placed at the top of the curb, as in No. 2. By making the quarter crown 0.24 ft. above the high gutter, a section is obtained with a slope even less than the author's, and with no double curb.

In Section No. 8 the worst conditions are encountered. The two gutters are made 0.36 ft. and 0.83 ft., respectively, and the quarter crown 0.16 ft. This gives as a section the upper dotted line, which is rather bad, but the speaker thinks that even here he would adopt it in preference to the double curb and the section shown by the author, having a slope of 0.37 ft. from the gutter to the center, and thus throwing the entire surface water to the low side of the street.

If, however, it is thought that the slope is too great, the speaker would reduce it by placing the step near the building, rather than at the curb line. The exact distance from the property line must be determined by the conditions. If, as is generally the case, a certain space is allowed to property owners for a display of goods, or for area-ways or courtyards, this step might be located at the edge of this space, with no inconvenience to any one. In this event the sidewalk would be laid on the same slope as before, but 0.5 ft. lower, and the cross-section of the street would be as shown by the lower dotted line, which the speaker thinks is much to be preferred to that used by the author.

If the latter plan were adopted, the upper sidewalk would extend some 3 or 4 ft. beyond the property line at the cross streets, and then return up the side street, running the grade out in a short distance and thus doing away entirely with the two steps at the curb. Should the gutters shown by the author be retained, the actual street would be improved by adopting the section shown by the lower dotted line.

If the above new sections should be used on a street where existing conditions were as described in the paper, no double curb would be required, and, for practical purposes, the cross-sections would be as good as those shown by the author. Mr. Tillson.

These results have been obtained by varying from the author's procedure in two ways: First, by making the crown on the quarter in every case where the difference in elevation of the curbs requires it; second, by not attempting to make the slope on both sides of the center uniform.

The speaker has used the first method for several years with uniformly good results, and is fully satisfied that, under the conditions considered in the paper, it is the proper course to adopt.

While the author has not always attained the results mentioned in the second reason, he has done so in Sections Nos. 5 and 9, and, apparently, has sought to in others, thus necessitating the use of the double curb, or a needlessly high step.

A. W. Hankinson, Jun. Am. Soc. C. E.—In determining crowns for asphalt pavements in New York City, that is, in the Borough of Manhattan, a standard section is assumed of $4\frac{1}{2}$-in. gutters, with a crown of $\frac{1}{100}$, and a quarter height of $\frac{1}{160}$ of the width of the roadway, thus making the proportion of crown to quarter as 8 : 5. When the curbs are not level, the gutter on the high side is deepened to a maximum of 9 ins., and if the gutters are still of unequal elevation, one-half of the difference of elevation is subtracted from the standard crown, in effect securing the standard fall of 2% from the crown to the gutter on the low side. It will be seen that in a 30-ft. roadway, with a difference of curb level of $11\frac{1}{2}$ ins., the result of following this plan will be a street with no crown at all, the pavement being on a straight grade from gutter to gutter. It seldom occurs that a greater difference in curb elevation is necessary on 30-ft. streets, as in most cases one of the curbs can be raised or the other lowered without doing very much damage to the adjoining property. The wider the roadway, the greater the difference in curb level that can be allowed; as in 60-ft. streets, the crown being $7\frac{1}{2}$ ins., a difference of 18 ins. can readily be adjusted. Mr. Hankinson.

The author, having the opportunity of adjusting not only the roadway, but also the sidewalk, it would seem that even his maximum difference of level of 1.65 ft. in 66 ft. could have been accommodated without the use of the stepped curb, by lowering the sidewalk on the high side. A step at the entrance of each store would be less objectionable than a continuous one at the curb for the length of the block.

Roadways of a greater width than 30 ft. have, in New York, usually a street railroad occupying 15 ft. of the street. When such railroad has no underground conduit to be used as a drain, the rails are placed on the contour of the pavement arch, as in a level 60-ft. section, the

Mr. Hankinson. center rails would be $\frac{1}{2}$ in. below the crown and the outside rails dropped $\frac{3}{4}$ in. from the center rails. As the pavement between the rails is crowned up 1 in., this $\frac{3}{4}$-in. drop amply provides for drainage. In the case of an underground trolley with slotted conduit, the slot rails are placed on the contour of the pavement crown and the tram rails made level if possible. Should, however, the difference of curb elevation be very great, the track is "terraced," that is, one track, that on the low side of the street, is dropped from 1 to 3 ins. from the other. By terracing the tracks and raising and dropping the outside rails $\frac{3}{4}$ in. on the high and low sides of the street, respectively, a difference of 19 ins. in curb level on a 60-ft. street has been adjusted without draining to one gutter.

The crowns used by the author give a much steeper cross-grade than could be safely used in New York City. His quarter height being quite high in proportion to the crown, a very sharp incline from quarter to gutter results. It may be that Wabash, Ind., is not subject to the fogs, mists and drizzling rains which make the New York pavements so slippery at times, and that a cross-grade of 3%, entirely unsuited to the climatic conditions obtaining in New York, would be eminently proper in an inland city.

HISTORICAL SKETCH

OF THE

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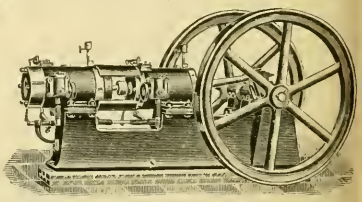

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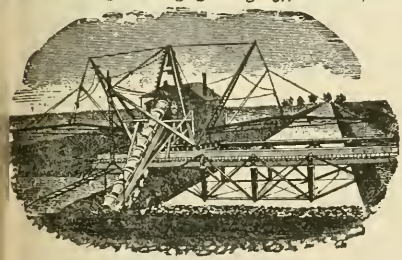

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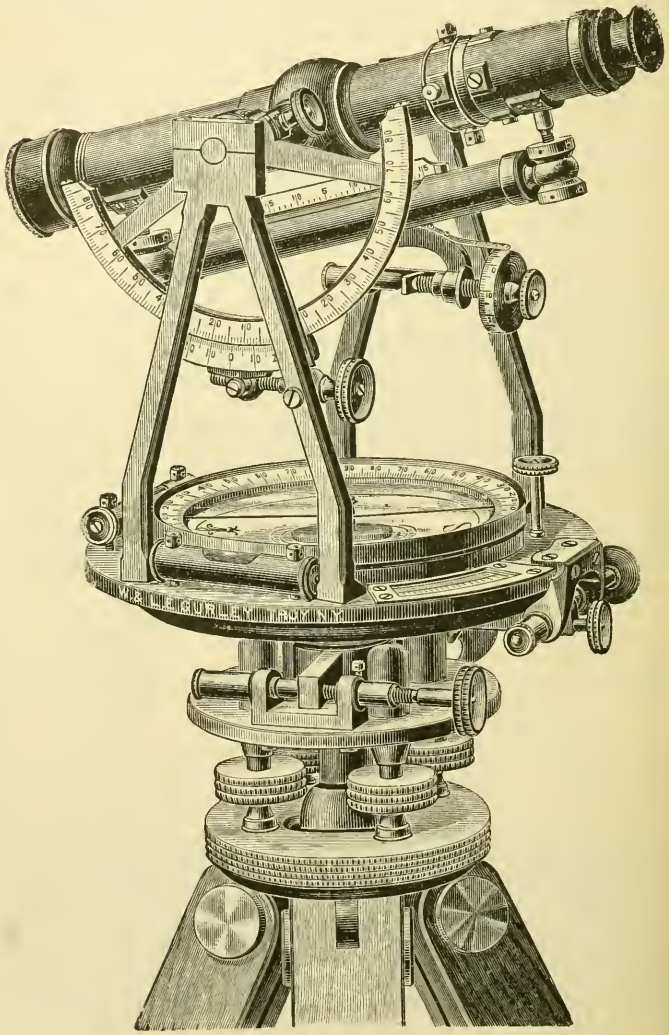

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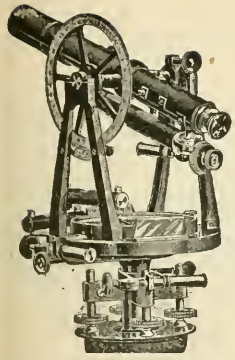

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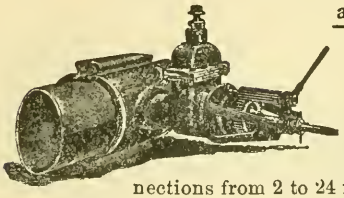

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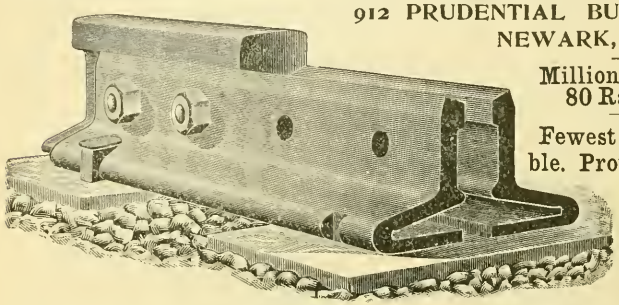

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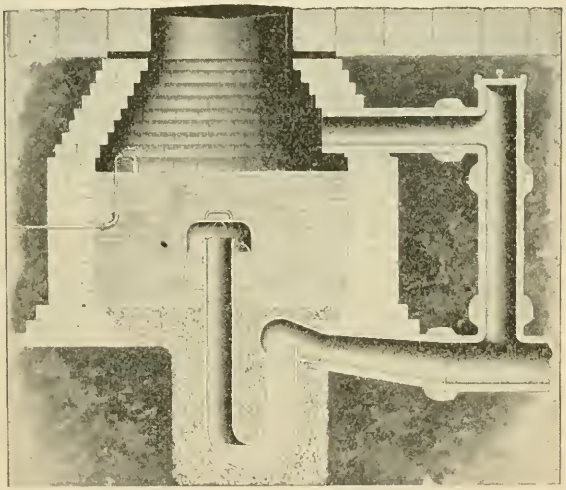

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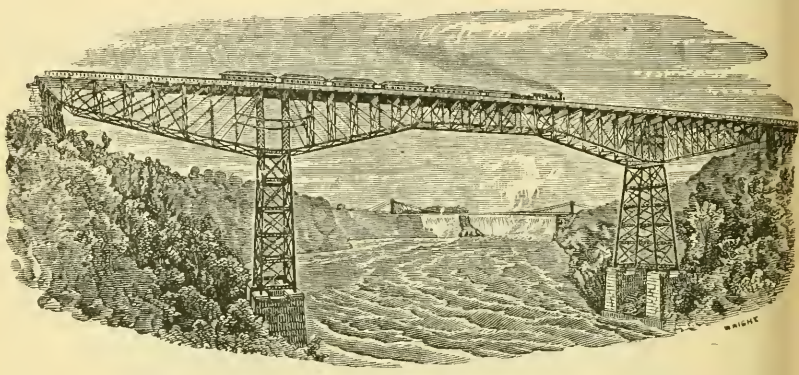

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PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publications.

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ON UNIFORM STANDARD TIME:—Sandford Fleming, Charles Paine, Theodore N. Ely.

ON ANALYSIS OF IRON AND STEEL:—Sub-Committee of the American Society of Civil Engineers (of the International Committee on Standards for the Analysis of Iron and Steel, of which Prof. J. W. Langley is Chairman)—Charles B. Dudley, William Metcalf, Thomas Rodd, A. E. Hunt.

ON UNITS OF MEASUREMENT:—George M. Bond, William M. Black, R. E. McMath, Charles B. Dudley, Alexander C. Humphreys.

ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Noble, George S. Webster, O. M. Carter, W. B. W. Howe, Louis C. Sabin, H. W. York.

The House of the Society is open every day, except Sunday, from 9 to 22 o'clock.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER, - - 533 Columbus
CABLE ADDRESS, - - "Ceas, New York."

Vol. XXV. APRIL, 1899. No. 4.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS.

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MINUTES OF MEETINGS.

OF THE SOCIETY.

April 5th, 1899.—The meeting was called to order at 21 o'clock, Mr. Allen Hazen in the chair; Charles Warren Hunt, Secretary, and present, also, 56 members and 8 visitors.

The minutes of the meetings of March 1st and 15th, 1899, were approved as printed in *Proceedings* for March, 1899.

The Secretary presented a paper by Professor L. M. Hoskins, entitled "General Criterion for Position of Loads Causing Maximum Stress in Any Member of a Bridge Truss," and also read a discussion on the subject by Frank H. Cilley, Esq.

The Secretary read a letter from the Building Code Commission of the City of New York inviting the Society to send a communication in regard to the existing building law, and its individual members to attend the public hearings on the matter, of which due notice is given in the *City Record.*

Ballots were canvassed and the following candidates declared elected:

As Members.

Arsène Perrilliat, New Orleans, La.
Charles Henry Rust, Toronto, Ont., Canada.
Jonas Waldo Smith, Paterson, N. J.
William Richardson Webster, Philadelphia, Pa.

As Associate Members.

Julius Leroy Adams, South Manchester, Conn.
George Baum, Bloomfield, N. J.
Edward Everett Betts, Chattanooga, Tenn.
Edward Wallace Bush, Hartford, Conn.
Frederick Albert Coleman, Rome, N. Y.
Allan Winter Cuddeback, Paterson, N. J.
Charles Adriance Mead, Newark, N. J.
James C Nagle, College Station, Texas.
George Fetter Stickney, Macon, Ga.

The Secretary announced that at the meeting of the Board of Direction, April 4th, 1899, the ballot on the reconsideration of Judd Allen Lockwood was canvassed, and Mr. Lockwood declared elected a Member of the Society.

Announcement was made that the following candidates were elected by the Board of Direction April 4th, 1899:

As Associates.

Alger Crocheron Gildersleeve, New York City.
John Byron Goldsborough, Croton-on-Hudson, N. Y.

As Juniors.

Pitson Jay Cleaver, Brooklyn, N. Y.
Thomas Crabb, Newark, N. J.
Almon Homer Fuller, Seattle, Wash.
Charles Rufus Harte, Boston, Mass.
Frederic de Peyster Hone, New York City.
Thaddeus Merriman, Washington, D. C.
Garret Edward Tilt, Paterson, N. J.

As Fellow.

John Robert Stanton, New York City.

The Secretary announced the following deaths:

Robert Delos Rowe, elected Member January 2d, 1890; died March 14th, 1899.

Arthur Sheppard Campbell Würtele, elected Member March 5th, 1873; died March 20th, 1899.

The Secretary announced that on Wednesday, April 12th, a meeting of the Juniors of the Society would be held, the subject for discussion being "The Duties and Responsibilities of an Inspector."

The Secretary announced the subject for discussion of the meeting of April 19th, 1899.

Adjourned.

April 19th, 1899.—The meeting was called to order at 20.45 o'clock, President Desmond FitzGerald in the chair, Charles Warren Hunt, Secretary, and present, also, 80 members and 11 visitors.

H. de B. Parsons, M. Am. Soc. C. E., described some novel methods of steam and hot water heating, and the subject was discussed by Messrs. W. H. Baldwin, H. W. Brinckerhoff and the President.

The Secretary presented a communication from T. H. McCann, M. Am. Soc. C. E., describing the laying and testing of two lines of 18-in. submerged water mains, and the subject was discussed by Messrs. L. B. Ward, O. H. Tripp, E. C. Moore, A. Potter, H. W. Brinckerhoff and the President.

The Secretary announced the following deaths: Jacob Blickensderfer, elected Member June 1st, 1881; died February , 1899, and Charles Irwin Brown, elected Member January 7th, 1891; died March 23d, 1899.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

April 4th, 1899.—Past President Thomas Curtis Clarke in the chair; Charles Warren Hunt, Secretary, and present also Messrs. Bensel, Morison, Owen, Thomson, Turner and Whinery.

The ballot on the reconsideration of the vote on Judd Allen Lockwood was canvassed and Mr. Lockwood was declared elected a Member of the Society.

The appointment by the President of a Committee of the Board of Direction, and a Local Committee of Arrangements for the Annual Convention was reported.*

A communication from the Building Code Commission of the City of New York inviting from the Society any communication in relation to the existing building laws of New York City was received.

* See p. 87.

The Secretary was instructed to reply to the same and to read the letter at the next meeting of the Society.

The Secretary reported that the late Herbert Steward, Assoc. Am. Soc. C. E., had bequeathed to the Society the sum of two thousand dollars ($2 000) to be known as the Herbert Steward Library Fund, and that his will also provided that the Society should receive such of the technical books in his library as it desires; these books to be selected by the Secretary.

The Secretary reported the successful negotiation with several publishers of technical books for the prompt donation to the Library of their publications, and that in future notices of such donations will be published in *Proceedings* under the heading "New Books of the Month."

The Secretary reported that on March 29th, 1899, the New York Electrical Society held one of its meetings in the Auditorium of the Society.

Applications were considered and other routine business transacted.

Two candidates for Associate, seven for Junior and one for Fellow were elected.

Adjourned.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society is open every day, except Sunday, from 9 to 22 o'clock.

MEETINGS.

Wednesday, May 3d, 1899, at 20.30 o'clock, a regular business meeting will be held. Ballots for membership will be canvassed, and a paper by John W. Hill, M. Am. Soc. C. E., entitled "Comparative Tests of Bituminous Steam Coals" will be presented. This paper is printed in this number of *Proceedings*.

Wednesday, May 17th, 1899, at 20.30 o'clock, a meeting of the Society will be held, the paper for the evening being "The Artificial Preservation of Railroad Ties by the Use of Zinc Chloride," by W. W. Curtis, M. Am. Soc. C. E. This paper is printed in this number of *Proceedings*.

Wednesday, June 7th, 1899, at 20.30 o'clock. At this meeting a paper by George W. Rafter, M. Am. Soc. C. E., entitled "On the Theory of Concrete," will be presented for discussion. This paper is printed in this number of *Proceedings*.

JUNIOR MEETINGS.

March 13th, 1899.—A meeting of Juniors was held for the purpose of discussing the advisability of inaugurating future special meetings of Juniors, Vice-President North in the chair; Charles Warren Hunt, Secretary. The Secretary reported a digest of the answers received to an interrogatory circular sent out for the purpose of ascertaining the views of all Juniors. It was concluded to test the matter by calling a meeting of Juniors for an informal discussion of some engineering topic, the Secretary being requested to fix a date for such meeting, and a subject for discussion.

April 12th, 1899.—A meeting of Juniors was held, Secretary Hunt in the chair, and present also twenty-five Juniors. The subject for the evening was, "The Duties and Responsibilities of an Engineer Inspector," and an interesting informal discussion occupying more than two hours was participated in by many of those present. The following resolution was passed:

Resolved, that it is the sense of this meeting that similar meetings of Juniors should be provided for, and that the Board of Direction be requested to authorize the holding of such meetings on the Second Wednesdays of May and June, 1899."

Messrs. J. M. Evans, M. E. Evans and E. D. Knap were appointed by the Chair an Executive Committee to act on behalf of the Juniors, if the proposed meetings should be authorized.

ANNUAL CONVENTION.

The Thirty-first Annual Convention will, as heretofore announced, be held at the Stockton Hotel, Cape May, N. J., June 27th to 30th, 1899.

The following Committees have been appointed to take charge of the Convention, and due notice of arrangements made for transportation, etc., will be issued.

COMMITTEE OF THE BOARD OF DIRECTION:

C. C. Schneider,
Henry G. Morse, Charles Warren Hunt.

LOCAL COMMITTEE OF ARRANGEMENTS:

L. Y. Schermerhorn,
A. Bonzano,
Joseph M. Wilson,
Samuel Rea,
Joseph T. Richards,
Theo. Voorhees,
L. M. Haupt,
John C. Trautwine, Jr.,
George S. Webster,
R. W. Lesley.

CURRENT TECHNICAL LITERATURE.

Under the heading "Monthly List of Recent Engineering Articles of Interest," will be found the second installment of the list begun in the March number of *Proceedings*. It will be seen that the number of periodicals to which reference is made has been largely increased, and that a general classification of engineering work on broad lines has been adopted.

The interest which has already been manifested by members of the Society in this list is gratifying, and several suggestions have been made in regard to it. It is the opinion of some that if the length and importance of each article, as well as whether it is illustrated or not, were stated, the usefulness of the list would be enhanced.

To be of any value an opinion in regard to each of the articles mentioned in this list would require the services of an expert in at least each of the classes into which the list is at present divided, and each article would, of necessity, have to be carefully studied by such expert. It seems probable that if the great value of some articles were pointed out, the authors and publishers of those not so treated might have some cause for complaint, even though the statement is made on the title-page of each of our publications that "This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications."

It is a well-known fact that a large percentage of the members of this Society are so situated as not to have ready access to technical libraries in which are deposited the various periodicals covered by this list, and that often those who have such access have not the time to look over each month all of these publications. It was with the intention of giving to each member a current knowledge of the literature on the special subject in which he is interested, and sufficient information to enable him to secure any special article for his library or scrap-book, that the list was started.

NEW BOOKS OF THE MONTH.

Four prominent publishers of scientific books have already agreed to donate to the library of the Society, promptly upon their issue, the books brought out by them. These newer books will be placed in a special alcove of the reading room, and monthly book notices, under the above heading, will be issued in *Proceedings* for the information of members. It is hoped that this list will also prove of value.

GEOGRAPHICAL DISTRICTS INTO WHICH THE SOCIETY IS DIVIDED FOR THE PURPOSES OF THE NOMINATING COMMITTEE.

The seven geographical districts into which the territory occupied by the membership has been divided by the Board of Direction, under Art. VIII, Sec. 1, of the Constitution, for the purposes of the Nominating Committee are herewith announced as follows:

District No. 1.—The territory within 50 miles of the Post Office in the City of New York.

District No. 2.—The States of Maine, New Hampshire, Vermont, Massachusetts, Rhode Island and Connecticut (except as included in District No. 1), and all countries in Europe.

District No. 3.—The States of New York and New Jersey (except as included in District No. 1), and the Dominion of Canada.

District No. 4.—The States of Pennsylvania, Delaware and Maryland and the District of Columbia.

District No. 5.—The States of Ohio, Indiana, Illinois, Michigan, Wisconsin, Iowa and Minnesota.

District No. 6.—The States of Virginia, West Virginia, North Carolina, South Carolina, Georgia, Florida, Kentucky, Tennessee, Alabama, Mississippi, Missouri, Arkansas and Louisiana, the West India Islands and all countries in South America.

District No. 7.—The States of North Dakota, South Dakota, Nebraska, Kansas, Texas, Montana, Wyoming, Colorado, Idaho, Utah, Washington, Oregon, California and Nevada; the following territories—Indian Territory, Oklahoma, New Mexico, Arizona and Alaska; the Republic of Mexico, and all countries in Central America, Asia, Australasia and Africa.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(March 15th to April 12th, 1899.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal.* Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings.* Eng. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal.* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal.* Western Soc. of Eng., Monadnock Block, Chicago, Ill.
(5) *Transactions.* Can. Soc. C. E., Montreal, Que., Can.
(6) *School of Mines Quarterly.* Columbia Univ., New York City, 50c.
(7) *Technology Quarterly.* Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Indicator.* Stevens Institute, Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 30c.
(10) *Cassier's Magazine.* New York City, 25c.
(11) *Engineering* (London). W. H. Wiley, New York City, 35c.
(12) *The Engineer* (London). International News Co., New York City, 35c.
(13) *Engineering News.* New York City, 15c
(14) *Engineering Record.* New York City, 12c
(15) *Railroad Gazette,* New York City, 10c.
(16) *Engineering and Mining Journal.* New York City, 15c.
(17) *Street Railway Journal,* New York City, 35c.
(18) *Railway and Engineering Review.* Chicago, Ill.
(19) *Scientific American Supplement.* New York City, 10c.
(20) *Iron Age.* New York City, 10c.
(21) *Railway Engineer.* London, England.
(22) *Iron and Coal Trades Review,* London, England.
(23) *Bulletin.* American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gaslight Journal.* New York City, 10c.
(25) *American Engineer.* New York City, 20c.
(26) *Electrical Review.* London, England.
(27) *Electrical World and Electrical Engineer.* New York City, 10c.
(28) *Industries and Iron,* London, England.
(29) *Journal,* Society of Arts, London, England.
(30) *Annales des Travaux Publics de Belgique.* Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des Ecôle Spéciales de Gand,* Brussels, Belgium.
(32) *Memoirs et Compt Rendu des Travaux.* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil.* Paris, France.
(34) *Portefeuille Economique des Machines.* Paris, France.
(35) *Nouvelles Annales de la Construction.* Paris, France.
(36) *La Revue Technique.* Paris, France.
(37) *Revue de Mecanique.* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways.* Paris, France.
(39) *Railway Master Mechanic,* Chicago, Ill.
(40) *Railway Age.* Chicago, Ill., 10c.
(41) *Modern Machinery.* Chicago, Ill., 10c.
(42) *Transactions.* Am. Inst. Elec. Eng., New York City, 50c.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal.* Military Service Institution, Governor's Island, New York Harbor, 75c.
(45) *Mines and Minerals.* Scranton, Pa., 50c.
(46) *Scientific American,* New York City, 10c.
(47) *Mechanical Engineer,* Manchester, England.

LIST OF ARTICLES.

Bridge.

Building a Concrete Arch. (40) March 3.
A Stone Arch Bridge of Moderate Cost. (13) March 30.
Three-Span Melan Arch Bridge Across the Passaic River, Paterson, N. J. (13) March 16.
The New York Tower Foundations. New East River Bridge. (14) April 1.
Screw-Pile Bascule Bridge. Caringa, India. (18) Feb. 25.
The Merits and Permanency of the Masonry Arch Bridge. Albert W. Buel. (9) April, 1899.

Bridge—(Continued).

Atchafalaya River Bridge Piers. (14) April 8.
Reconstruction of the Penrose Ferry Bridge, Philadelphia. (14) April 8.
Bridge 69, New York Division, Pennsylvania Railroad. (14) March 25.
Chicago, Madison and Northern Swing Bridge Over Drainage Canal. (18) March 4.
The Inkissi Bridge, Congo Railway. (12) March 3.
Bending Moments in Simple Beams Due to Wheel Loads. Thomas J. Brereton. (6) Jan., 1899.
Note sur le Calcul du Pont Vierendeel. E. Haerens. (31) Oct.-Dec., 1898.
Le Pont-Canal de Biare, sur la Loire. E. Rouyer. (33) March 11.
Note sur la Construction du Pont Alexandre III. MM. Resal et Alby. (43) Fourth trimestre, 1898.
Notice sur le Pont a Transbordeur de Bizerte. M. Picard. (43) Fourth trimestre, 1898.

Electrical.

The Trend of Designing in Central Electric Stations. B. J. Arnold. (24) Feb. 20.
Systems of Electric Rates. E. L. Debell. (24) Feb. 6.
Artificial Lighting (Gas and Electric) for Public Institutions. (24) Jan. 30.
Electric Installations for Lighting and Power on the Midland Railway, with Notes on Power Absorbed by Shafting and Belting. W. E Langdon. (21) Feb., 1899.
The Lighting of Country Residences. (14) April 1.
The Public Supply of Electricity at Canterbury (Eng.). (12) March 17.
Two New English Direct-Current, High-Voltage, Three-Wire Central Stations. (27) March 25.
The Difference Between Good and Bad Incandescent Lamps. Francis W. Willcox. (27) March 18.
The Growth of Electric Lighting in New York. (27) March 18.
Nernst's Electric Light. James Swinburne. (29) Feb. 10. (27) Feb. 25.
The Relation Between Mean Spherical and Mean Horizontal Candle-Power of Incandescent Lamps. Prof. R. A. Fessenden. (27) Feb. 25.
The Electric Lighting of Gibraltar. (27) Feb. 18.
Canterbury Corporation Electricity Works. (26) March 10.
The City of London Electric Lighting Company's Works. (26) March 3.
Multiplex Telegraphs. (15) March 17.
The Use of the Wehnelt Interrupter. Louis S. Levy. (27) March 25.
Observations on the Wehnelt Interrupter. George T. Hanchett and Charles T. Child. (27) March 25.
Experiments with the Wehnelt Interrupter. George T. Hanchett. (27) April 8.
Electricity Direct from Coal. A. J. Rogers. (28) March 10.
Wireless Telegraphy. G. Marconi. (28) March 10. (13) March 30.
The Isolated Plant of the New Boston Railway Terminal. (27) March 11.
Alternating Currents Twenty-five Years Ago and To-Day. Edwin J. Houston and A. E. Kennelly. (27) March 4.
The Meran-Bozen Transmission. (27) March 4.
Storage Batteries and Railway Power Stations. Robert McA. Lloyd. (27) Feb. 25.
Electric Power Stations of Granada, Spain. (27) Feb. 25.
Electricity on Board Ship. S. Dana Greene. (27) Feb. 18. (42) Jan., 1899.
Electric Elevators at Drury Lane Theatre. (26) Feb. 3.
Tonawanda Transformer Station. Orrin E. Dunlap. (27) March 18.
Transmission Plant of the Southern California Power Company. (27) March 18.
Water and Electric Power Plant near Helena, Mont. (27) March 11.
Motor-Dynamos or Accumulators. Alton D. Adams. (17) April, 1899.
Electric Power in Steel Making. Eugene B. Clark. (10) April, 1899.
Notes on Telegraphy and Telephony. W. H. Freedman. (6) Jan., 1899.
Les Accumulateurs Électriques à Gaz. (36) March 25.

Marine.

American Rivalry in the Shipbuilding Industry. (22) Jan. 20.
A Process of Fireproofing Wood for the Woodwork of War Ships. (13) March 23.
Ships' Refrigerating Plants—Linde System. (19) March 25.
The New Cruiser *Chitose*, for Japan. (19) March 18.
Submarine Survey. Charles Bright. (19) March 18.
The Lake Freight Steamer *Troy*. (13) March 30.
H. M. Battleship *Glory*. (11) March 17.
The Japanese Armored Cruiser *Asama*. (11) March 3.
The Russian Armored Cruiser *Gromovoi*. (12) March 24.
The Logical Arrangement of the Motive Power of Warships. Com. George W. Melville. (12) March 24.
Russian Cruiser *Pollada*. (12) March 10.
Argentine Cruiser *General Belgrano*. (12) March 3.
The Monitor, the Battle-Ship, the Cruiser and the Destroyer. G. W. Dickie. (9) April, 1899.
French Submarine Torpedo Boats. (46) April 8.
Engines of the *S. S. Ultonia*. (11) March 3.
Institution of Naval Architects Meeting. (11) March 31. (12) March 31. (28) March 30.
Ice-Breaking Steamer *Ermack*. (11) March 31.
The French Passenger Steamer *Laos*. (11) March 31.
Engines of the Allan Liner *Castilian*. (12) March 31.

Marine—(Continued).

Japanese Cruiser *Kasagi*. (12) March 31.
Remarks on the Increasing Frequency of Failure of Propeller Shafts. Frank Caws. (28) March 30.
The 150-Ton Revolving Derrick, Newport News Shipbuilding & Dry Dock Co. (18) March 18.
Le Cuirassé Anglais le *Goliath*. M. Hachebet. (33) March 11.
Une Visite au *Goubet, No. 2*. (36) March 10.

Mechanical.

A Smoke-Preventing Furnace Door. (12) March 24.
Boiler Plant, C. R. I. & P. Ry. (18) March 4.
The Hutchinson Sectional Vertical Tube Boiler. (20) Jan. 19.
Recent Advances in the Manufacture of High Grade Boiler and Pipe Materials. (13) March 23.
Recent Practice in Steam Super-heating. (13) April 6.
Circulation and Heat Transmission in Water-Tube Boilers. David R. Todd. (47) April 1.
Steam Pipes. J. T. Milton. (12) March 24.
Steam Mains for Electric Supply Stations. R. Summerfield. (11) March 10.
The Measure of Horse-Power. (12) March 10.
Novel Rotary Engine. (18) March 18.
Two Power Multiplying Engines. (13) April 6.
Steam Turbines. Rankin Kennedy. (26) Feb. 17.
Cooling Tower and Condenser Installation. J. H. Vail. (20) Feb. 2.
Simm's Patent Motor for Launches. (28) March 17.
Compound Blowing Engines for the Acklam Furnaces. (11) March 17.
Fairbanks-Morse Direct Connected Gasoline Engine and Air Compressor. (25) April, 1899.
Gas Engines. (14) April 8.
A New Oil Motor. (12) March 24.
A Factory Power Plant. (14) April 8.
Mechanical Plant of the Boston Southern Terminal Station. (14) March 18.
New Power Plant of the Consolidated Traction Co., Pittsburg. (15) March 24.
The Paderno-Milan Power Transmission. (18) Feb. 25.
The Design of Roughing Rolls. William Hirst. (20) March 16.
New Continental Rolling Mill Equipment. (22) Jan. 6.
Thiesen's Apparatus for Purifying Blast-Furnace Gas and for Collecting Coke-Oven By-Products. (22) Feb. 3.
Hot-Blast Valves. James Robinson. (22) Feb. 17.
The Doherty Iron-Castings Process. (22) March 3.
Testing Machines. F. A. Riehle. (18) April 1.
A Decade of Progress in Reducing Costs. C. Kirchhoff. (20) March 2.
Electrically Driven Air Pumps. (11) March 10.
Electric Pumps and Pumping. A. W. K. Pierce. (22) Feb. 17. (41) April, 1899.
The Lecomte System of Bunsen Burner Construction. (24) Jan. 16.
Ice Machinery in West Africa. (12) March 3.
English and American Machine Tools. T. Wood Meakin. (41) April, 1899.
Machine-Shop Management in Europe and America. H. F. L. Orcutt. (9) April, 1899.
Depreciation as an Element of Machine-Shop Cost-Keeping. H. M. Norris. (9) April, 1899.
The Development of Wood-Working Machinery. John Richards. (9) April, 1899.
The Early Use of Rolls in the Manufacture of Metals. W. F. Durfee. (10) April, 1899.
Report of Committee on Acetylene Generators. (29) Feb. 24.
Acetylene. (11) March 17.
The Real Value of Acetylene as an Illuminant. (12) March 31.
The Castner-Kellner Alkali Works at Weston Point. (26) March 31.
Motor Vehicles for Heavy Loads. (12) March 3.
Mr. Lanchester's Motor Carriage. (12) March 17.
New Features in Moto-Vehicle Design. (12) March 17.
A German Electric Omnibus. (18) March 25.
Details of Serpollet Steam Motor. (28) Feb. 24.
Proposed Standard Metrical Sizes for French Motor Car Driving Chains. (28) March 10.
Edinburgh Cycle and Motor Car Exhibition. (28) Feb. 17.
The Motor Vehicle in Commercial Operation. G. Herbert Condict. (27) March 4.
The Conget Variable Speed Gear for Motor Tricycles. (28) Feb. 24.
Some New Features in Motor Vehicle Design. Thomas H. Parker. (28) March 3.
The Application of Steam to Self-Contained Road Vehicles. (28) Feb. 17.
A New Type of Motor Vehicle. (28) Feb. 24.
Liquid Fuel. Sir Marcus Samuel. (29) March 17.
The Application of X Rays to the Examination of Fuels. (22) Feb. 24.
The Yard-Grading of Pig Iron. William B. Phillips. (28) Feb. 3.
Repèchage d'un Titan à Mostaganem. G. Prouteau. (36) March 25.
Grue Electrique de 50 Tonnes. (36) March 25.
Sur un Pyrometre Portatif. H. Schoentjes. (31) Oct.-Dec., 1898.
Défense de la Théorie Pratique de la Machine à Vapeur. Dwelshauvers-Dery. (37) Jan., 1899.
La Compression de la Vapeur dans l'espace mort. L. Anspach. (37) Jan., 1899.

Military.

High Explosives in Guns and Shells. (20) Feb. 23.
Quick-Firing Field Gun Carriage. (12) March 10.
Vicker's 12-In. Breech Loading Gun. (12) March 10.

Mining.

An Account of the Maypole Colliery Sinking, Wigan. (22) Jan. 13.
The Rating of Coal Mines. Edward Boyle. (22) Jan. 27.
Modern Pumping Machinery for Mine Service. Otto H. Mueller. (10) April, 1899.
Notes on Michigan Mining Practice. W. L. Hildburgh. (6) Jan., 1899.
Long Wall Advancing, Compared with Room and Pillar. Edward Jones. (45) April, 1899.
Rhodesia and Its Mines in 1898. William Fischer Wilkinson. (29) Feb. 3.
Cornish Mines and Cornish Miners. (29) March 10.
A New Method for the Determination of Zinc. A. C. Langmuir. (28) March 10.
The Semet-Solvay By-Product Coke-Oven Plant at Ensley, Ala. (13) March 16.
Traction Electrique par cables sans fin dans les Mines Montceau-Fontaine (Belgique). H. Schmerber. (33) March 25.

Municipal.

The Massachusetts State Highway Report for 1898. (14) March 25.
Tests of Paving Brick at the Iowa State College. (14) April 1.
The New Gas Works at Edinburgh. (12) March 24.

Railroads.

Speed of Express Trains in France. (15) April 7.
Notes on Colorado Railways. (13) April 6.
The Nilgiri Mountain Railway. (12) March 31.
The New Terminal of the Orleans Railway at the Quai d'Orsay. Jacques Boyer. (9) April, 1899.
The Passy Underground Railroad. (19) March 25.
The Railway Systems of Central and South America. (13) March 23.
The Trans-Siberian Railway and the Coal Resources of Siberia. (22) Jan. 27.
Sand Blast for Removing Paint from Tender Tanks. (25) March, 1899.
A New High-Speed Locomotive. (19) April 1
Eight-Wheel Class H. Passenger Locomotive, Wabash Railroad. (15) March 24.
Ten-Wheel Passenger Locomotive, Buffalo, Pittsburg and Rochester Railway. (15) March 31. (39) April, 1899. (18) March 25. (13) March 23.
Consolidation Compound Locomotive, Northern Pacific Railway. (18) April 1.
Schenectady Moguls for the Southern Pacific. (15) March 31.
Vauclain's Compressed Air Locomotives with Heater. (25) Feb., 1899.
The Heisler Geared Locomotive. (40) March 24.
Consolidation Locomotive for the Lehigh Valley Railroad. (40) March 10.
Express Passenger Engine, Lancashire and Yorkshire Railway. (12) March 17.
Erecting a Locomotive. Prof. R. A. Smart. (41) April, 1899.
Tests on Wide and Narrow Fireboxes. H. H. Vaughn. (25) April, 1899.
Cast-Steel Side Rods, Philadelphia and Reading Railway. (25) April, 1899.
Powerful Passenger Locomotives on Fast Trains. (25) Feb., March, 1899.
Underhung Driving Spring Rigging. L. R. Pomeroy. (25) Feb., 1899.
Steam Motor Car on the P. C. C. and St. L. (15) March 31.
Water Tubes in Locomotive Fireboxes, London and South Western Railway. (25) March, 1899.
Some Cast-Steel Locomotive Details. A. T. and S. F. Ry. (25) March, 1899.
Compound Express Locomotive, Austrian Stadtsbahn. (12) March 31.
Steam and Electric Mine Locomotives. (16) April 8.
An "Atlantic Type" Locomotive in Germany. (15) March 17.
Locomotive Coal Tests, C. C. C. & St. L. Ry. and Purdue University. (25) Feb., 1899.
Anthracite Coal Burning, N. Y., O. & W. Ry. (25) March, 1899.
80 000-lb. Capacity Gondola Car for the Lake Terminal Road. (15) March 31.
60 000-lb. Freight Car Trucks of the Cleveland, Cincinnati, Chicago and St. Louis. (15) March 24.
66-Foot Car for Steel Rails. (25) April, 1899.
Light and Heavy Cars. O. H. Reynolds. (25) April, 1899.
Scales for Weighing Locomotives. Baldwin Locomotive Works. (25) April, 1899.
Permanent Way for Steam Railways. (13) March 23.
The First T-Rails Ever Rolled. (23) Feb. 15.
Rail Trimming on the Michigan Central and Santa Fé Roads. (18) March 4.
Side Scraper and Leveler, Yazoo and Mississippi Valley Railway. (18) March 25.
Steel Water Tanks, A. T. & S. F. Ry. (39) April, 1899.
New Car Shops, M. K. & T. Ry., at Sedalia, Mo. (39) April, 1899.
A Yard Installation for Testing Air Brakes. (15) March 24.
The Resistance of Loaded and Empty Freight Cars. (15) March 24.
The Baldwin Train Resistance Formula. (15) March 17.
Freight Loading System at Richmond, Va. (15) March 24.
Gross Receipts per capita and per mile of Track in Principal American Cities. (17) April, 1899.
Rapid Transit in Dense Centers of Population. John Lundie. (17) April, 1899.
Power Station and Repair Shops of the Cleveland Electric Railway Company. (17) April, 1899.

Railroads—(Continued).

Notes on Operation of a Light Mineral Railroad. James Douglas. (18) April 1.
The Movable Platform of Saint Ouen. (19) April 8.
Emergency Single Track Blocking. (15) March 24.
Block Signaling on the Atchison, Topeka and Santa Fé Railway. (13) March 30.
The Distant Signal. E. D. Wileman. (18) Feb. 25.
Inspection of Automatic Block Signals. (15) March 17.
Automatic Block Signals on the Southern Pacific at Los Angeles. (15) March 17.
The Lewiston, Brunswick and Bath Street Railway. (24) March 4.
Chicago and Milwaukee Three-Phase Electric Railway. (27) April 8.
Commercial Aspects of Electric Traction in Great Britain. Emile Garcke. (9) April, 1899.
Glasgow Corporation Tramways. H. F. Parshall. (11) March 24.
Madrid Electric Tramways. (26) Feb. 24.
Barcelona Electric Tramways. (26) Feb. 24.
Appareil permettant de poser Mécaniquement les Voies Ferrées. P. F. Dubois. (43) Fourth trimestre. 1898.
Appareil Dynamométrique de M. Téodorovitch. Henri Chevalier. (32) Jan., 1899.
Les Tramways Electriques du Puy (Haute-Loire). (34) March. 1899.
Le Chemin de Fer Métropolitain de Paris. J. Hervieu. (35) March, 1899.
Le Chemin de Fer de Courcelles à Passy, et au Champ de Mars. (36) March 10.
Note sur un Wagon á Platforme surbaissée Destiné au Transport des grosses piéces mécaniques. J. Koechlin. (38) March, 1899.

Sanitation.

Sewer Catch-Basin and Cleaning Apparatus at Wilmington, Del. T. Chalkley Hatton, M. Am. Soc. C. E. (13) March 30.
The North Yarra Sewer Tunnel, Melbourne. (14) April 8.
New Units in the Design of Sewers. (14) March 25.
The Pollution of the Passaic River. (14) March 18.
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Studies for a High Level Gravity Sewer for Boston and Vicinity. (13) March 16.
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Some Engineering Features of a Boston School. (14) March 25.
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The Acker, Merrall & Condit Stables, New York. (14) March 18.
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The Buffalo General Hospital. (14) April 8.
The New Iron Foundry of the General Electric Company. (15) April 7. (20) April 6.
Le Palais des Mines et de la Metallurgie á l'Exposition et le metal deployé. (36) March 10.
Des Derniers Progrès Accomplis dans les Constructions en Ciment Armé. N. de Tedesco. (32) Jan., 1899.

Water Supply.

A Gasoline Pumping Plant for the Water-Works of Toms River, N. J. (13) March 30.
Ice Shields at the Buffalo Water-Works. (14) April 1.
New Water Tower at Schenectady, N. Y. (14) April 1.
Proposed State Legislation to Prevent Water Pollution. (13) March 16.
The Transfer of the Watertown Water-Works. (14) March 18.
The New High-Service System, Lawrence, Mass. (14) March 25.
The New Rockford Ill. Pumping Plant. (14) March 18.
Standpipe Failure at Murphysboro, Ill. (14) March 18.
The Coolgardie Steel Pipe Contract. (22) Jan. 13.
The Montrose Irrigation Canal, Colorado. W. P. Hardesty. (13) March 23.
Water Softening. W. N. Twelvetrees. (10) April, 1899.
The Periar (India) Dam. (19) April 8.
Repairing a Deep-Well Pump. (12) March 31.
Installation Hydro-Electrique de Paderno d'Adda. Transport de force de 13,000 chevaux á 33 kilometers. E. Vannotti. (33) March 25.
Note sur une Etude Experimentale de l'Action de l'Eau sous Pression sur les Maconneries. P. Breuillé. (43) Fourth trimestre. 1898.
Résolution de Deux Questions sur le Mouvement Varié des Eaux (Propagation d'une Crue et d'une Maree Fluviale). E. Haerens. (30) Feb., 1899.
Expériences sur le Mouvement de l'Eau dans les Canaux Rectangulaires Alimentés par une Vanne. F. Keelhoff. (31) Oct.-Dec., 1898.

Waterways.

The Problem of the Mississippi. Herman Haupt. (3) April, 1899.
Expériences sur la Resistance de Fermettes de Barrages établies pour soutenir des Aiguilles et mode de consolidation adopté pour permettre l'application des Vannes. M. Claise. (43) Fourth trimestre, 1898.
Role Nouveau du Dragage Mécanique des Passes dans la Régularisation des Fleuves. V. E. de Timonoff. M. Am. Soc. C. E. (43) Fourth trimestre, 1898.
Note sur le Remplacement du Pivot de la Porte d'ebe aval, rive droite, de l'écluse de l'ouest du Port de Dunkerque. M. Perrier. (43) Fourth trimestre, 1898.
Essai sur les Travaux de Fascinages et la Construction des Digues ou Description du Reendiguement des Polders du Bas-Escaut Belge. U. N. Kümmer. (30) Feb., 1899.
Rapports sur les Travaux de la 2e Section (Complement) et de la 5e Section du Congres International de Navigation. E. Walin. (31) Oct.-Dec., 1898.
Les Travaux d'Amelioration du Port du Havre. Georges Leugny. (36) March 25.
Distribution des Vitesses Suivant la Verticale dans les courants de Marée. M. Bourdelles. (43) Fourth trimestre, 1898.
Installation Électrique pour l'alimentation du canal de Bourgogne. M. Galliat. (43) Fourth trimestre, 1898.

NEW BOOKS OF THE MONTH.

Unless otherwise specified, books in this list have been donated to the Library by the Publisher.

LUBRICANTS, OILS AND GREASES.

Treated Theoretically and Giving Practical Information regarding their Composition, Uses and Manufacture: A Practical Guide for Manufacturers, Engineers, and Users in general of Lubricants. By Iltyd I. Redwood. Cloth, 9 x 5 ins., 54 pp. plates. New York, Spon & Chamberlain, 1898. $1.50.

The author has endeavored to give engineers an insight into the properties of the various lubricants, and to explain the theory and general requirements that govern their manufacture. The contents are: Part I, Theoretical: Part II. Manufacture. There is an index of four pages.

MINERAL OILS AND THEIR BY-PRODUCTS.

Including a Short History of the Scotch Shale Oil Industry, the Geological and Geographical Distribution of Scotch Shales, Recovery of Acid and Soda Used in Oil Refining, and a List of Patents relating to Apparatus and Processes for Obtaining and Refining Mineral Oils. By Iltyd I. Redwood. Cloth, 9 x 6 ins., 336 pp., illus. New York, Spon & Chamberlain, 1897. $6.00.

The local importance of the Scottish Shale Oil Industry has received recognition in practical treatises on petroleum, but the author has attempted to deal with the subject more fully than there has been opportunity for in works of a comprehensive character. There is an index of ten pages.

PORTLAND CEMENT: ITS MANUFACTURE AND USE.

By Charles D. Jameson, M. Am. Soc. C. E. Cloth, 9 x 6 ins., 192 pp., 7 pl. New York, D. Van Nostrand Company, 1898. $1.00.

This monograph is the outgrowth of a course of lectures delivered to engineering students. The contents are: General Considerations; Historical Data; Manufacture of Portland Cement; Testing Cements; Abstracts from American, German, French and Austrian Specifications for Portland Cement; The Chemical Processes Concerned in the Hardening of Hydraulic Cements; Cement Testing Machines and Stone Crushers; The Use of Portland Cement; Cement Tests. There is an index of six pages.

THE BAROMETRICAL DETERMINATION OF HEIGHTS.

A Practical Method of Barometrical Leveling and Hypsometry, for Surveyors and Mountain Climbers. By F. J. B. Corderio. Roan, 7 x 5 ins., 28 pp. New York, Spon & Chamberlain, 1898. $1.00.

The author, in the preface, says: "The discrepancies arising in the calculation of mountain heights by the barometrical formulas which have hitherto been in use have brought this valuable, and in many cases only applicable, method into disrepute. The fault has lain in the formulas, not in the method, which is one susceptible of great accuracy." The author furnishes new formulas.

ENERGY AND HEAT.

By John Roger. Cloth, 7 x 4 ins., 36 pp. New York, Spon & Chamberlain, 1899. 50c.

The author contends that the term "latent heat," the measurement of the energy of the thermal unit, and the accepted relations of heat and energy are fallacious or misleading, and that the economy of the steam engine is much underestimated.

TRANSFORMER DESIGN.

A Treatise on Their Design, Construction and Use. By George Adams. Second Edition. Cloth, 7 x 5 ins., 75 pp. New York, Spon & Chamberlain, 1899. $1.50.

The author states that he has endeavored to avoid historical matter and the use of mathematical analysis, in most cases giving results only or single formulas. The contents are: Types of Transformers; Power Factor; Induction and Iron Losses; Induction and Copper Losses; Sine Curves; Thickness of Core Plates; Distribution; Lamppost Transformer; The Three Wire System, etc. There is an index of four pages.

THE PURIFICATION OF PUBLIC WATER SUPPLIES.

By John W. Hill, M. Am. Soc. C. E. Cloth, 9 x 6 ins., 304 pp. New York, D. Van Nostrand Company, 1898. $3.00. (Donated by the author.)

This work is intended to present the facts and causes of pollution of sources of public water supply, the effect of this pollution on the typhoid rates of the larger cities, and to show that the typhoid rates have been reduced by the introduction of water from purer natural sources and by filtration of polluted waters.

The headings of chapters are: Sources of Public Water Supply; Bacterial Contents of Various Waters; The Typhoid Bacillus and Typhoid Fever; Classification of Cities by Typhoid Fever Statistics; Pure and Purified Waters; Citations on Typhoid Fever Epidemics; Sedimentation of Polluted Waters; Sterilization of Drinking Water; Filtration of Water Supplies; Types of Sand Filters; Mechanical Filters; Hamburg Settling-Basins and Filters; The Filters of the Berlin Water-Works; The Fischer Filter and Anderson Purifier; Filters Proposed for Cincinnati; Cost of Filters and Filtration; Typhoid Fever Statistics from Large Cities of the World; The Bacteria; The Legal Liability of Cities and Water Companies for Damages by Sewage Polluted Water. Ten pages are devoted to an index.

THE PURIFICATION OF SEWAGE.

Being a Brief Account of the Scientific Principles of Sewage Purification and Their Practical Application. By Sidney Barwise, M.D. Cloth, 7 x 5 ins., 150 pp. New York, D. Van Nostrand Company, 1899. $2.00.

The question is dealt with chiefly from a chemical and biological standpoint. It has been the endeavor of the author to present the latest researches upon the subject.

The headings of chapters are: Sewage; Its Nature and Composition; The Chemistry of Sewage; Varieties of Sewage and the Changes It Undergoes; River Pollution and Its Effects; The Land Treatment of Sewage; Precipitation, Precipitants and Tanks; Filtration or Nitrification; Special Forms of Sewage Filters; The New Departure, "Bacteriolysis." Four pages are devoted to an index.

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DEATHS.

BLICKENSDERFER, JACOB.....Elected Member June 1, 1881; died Feb. , 1899.

BROWN, CHARLES IRWIN.....Elected Member Jan. 7th, 1891; died March 23d, 1899.

ROWE, ROBERT DELOS.....Elected Member Jan. 2d, 1890; died March 14th, 1899.

WÜRTELE, ARTHUR SHEPPARD CAMPBELL.....Elected Member March 5th, 1873; died March 20, 1899.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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COMPARATIVE TESTS OF BITUMINOUS STEAM COALS.

By JOHN W. HILL, M. Am. Soc. C. E.

TO BE PRESENTED MAY 3D, 1899.

During the past few years several papers have appeared in the *Transactions* of the American Society of Mechanical Engineers, and in engineering periodicals, touching the ultimate chemical analysis of coals, especially bituminous and semi-bituminous coals, and upon the determination of the heating power of coals from the proximate and ultimate analyses and by calorimetric methods; and an attempt has been made upon more than one occasion, to deduce from a knowledge of the intrinsic thermal value of a coal the relation between its calorific power and the practical results which can be obtained from it when burned under steam boilers. In one of the most interesting and exhaustive of these papers, contributed by the late Charles E. Emery, M. Am. Soc. C. E., the whole field of experimental information on the comparative efficiencies of coals is briefly and clearly reviewed.*

The papers contributed by Professor R. C. Carpenter, of Cornell University; Mr. George H. Barrus, M. E., of Boston; Professor N. W.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in *Transactions*.

* "Comparative Tests of Steam Boilers with Different Kinds of Coal." *Transactions*, American Society of Mechanical Engineers, Vol. xvii.

Lord, of the Ohio State University, and others, when taken together, seem to indicate a more profound interest in the composition, energy and practical possibilities of steam coals, than has been displayed in all the years that have transpired since Professor Walter R. Johnson made his celebrated investigations for the United States Navy,* and suggest that any experimental data upon this subject should be welcome to those whose interests touch, however indirectly, this important factor of modern civilization.

During the summer of 1897, the author was commissioned by the Honorable Board of Administration, of Cincinnati, to investigate and report on the various bituminous steam coals available in the local market, with a view to obtaining a set of factors by means of which the prices asked by dealers for different kinds and grades of coal could be reduced to a uniform basis for the comparison of bids. The practical application of these factors can best be illustrated by the fact that heretofore it has generally been assumed, upon receiving bids on coal for the City Water Department or any other municipal service, that there was no large difference in the quality of the various coals offered, and that the lowest bid represented the most economical purchase. Each mine owner or agent very naturally has insisted that his coal was equal or superior to that of his competitors, from which the average city official would argue that if one coal is quite as good as another, the lowest bid represented the greatest economy to the corporation. By means of the factors mentioned, the price bid is multipled, and a new set of prices obtained for purposes of comparison only; and that price, which after such reduction is the lowest, represents the most economical coal for steam purposes.

The experimental work embraced:

First.—A practical steam test of each kind and grade of coal (of which there were over 50), under uniform conditions, for a period of 16 hours.

Second.—A proximate chemical analysis of each kind of coal.

Third.—A calorimetric determination of the heating power of each kind of coal.

The investigations, with some interruptions, occupied the interval from July 27th, 1897, to April 28th, 1898, the evaporation tests being conducted at the Hunt Street Pumping Station of the City Water-Works, and the laboratory work in the author's office in the Glenn

* Washington Navy Yard, 1842-43.

Building. In the physical tests, from 18 000 to 25 000 lbs. of coal were burned during a continuous trial of 16 hrs., and from 168 000 to 178 000 lbs. of water were supplied to the boilers.

The duration of the physical tests, 16 hrs., was fixed upon in order to embrace two full watches or tricks of the engineers, firemen and coal wheelers employed at the station. All the different kinds of coals were fired under the same boilers by the same firemen, one fireman firing during the 8-hour morning watch, and the other during the 8-hour evening watch, the relative position of the men being changed at the end of each series of eight tests.

The coals were burned in a pair of Galloway fire-box boilers, Fig. 1, built by the Edge Moor Iron Company, of Wilmington, Del. These boilers had been in service at the station for ten years prior to their use for the coal tests, but were in excellent order, and during the period of these tests were used for no other purpose.

The physical tests contemplated the same amount of work per hour for each kind and grade of coal, the amount of coal burned to perform this work to be varied according to its quality. With some of the inferior coals, especially with "Slack," this could not always be accomplished, and the rate of steam production was therefore lowered slightly to admit of burning such coals under fair conditions. The steam was used in operating the pumping engines in the station, which were under the author's control, in order that the consumption of steam should keep pace with the rate of production adopted for the tests. The whole work of the station during the interval of the physical tests was nicely adapted to the requirements and conditions of the coals submitted to investigation.

To avoid as much as possible the furnishing of selected samples of coal for test purposes, the city purchased the coal under the formal requisitions of the Water Department, and the dealers were in no instance permitted to be present during, or have knowledge of the results of, the trials. It was hoped thus to arrive at the fair commercial value of the respective coals, and to eliminate from the economic results the influence of samples selected for the occasion.

The two firemen who stood all watches during the tests were detailed by the Superintendent of Water-Works for this special duty. All assistance specially required for the tests was furnished from the author's office.

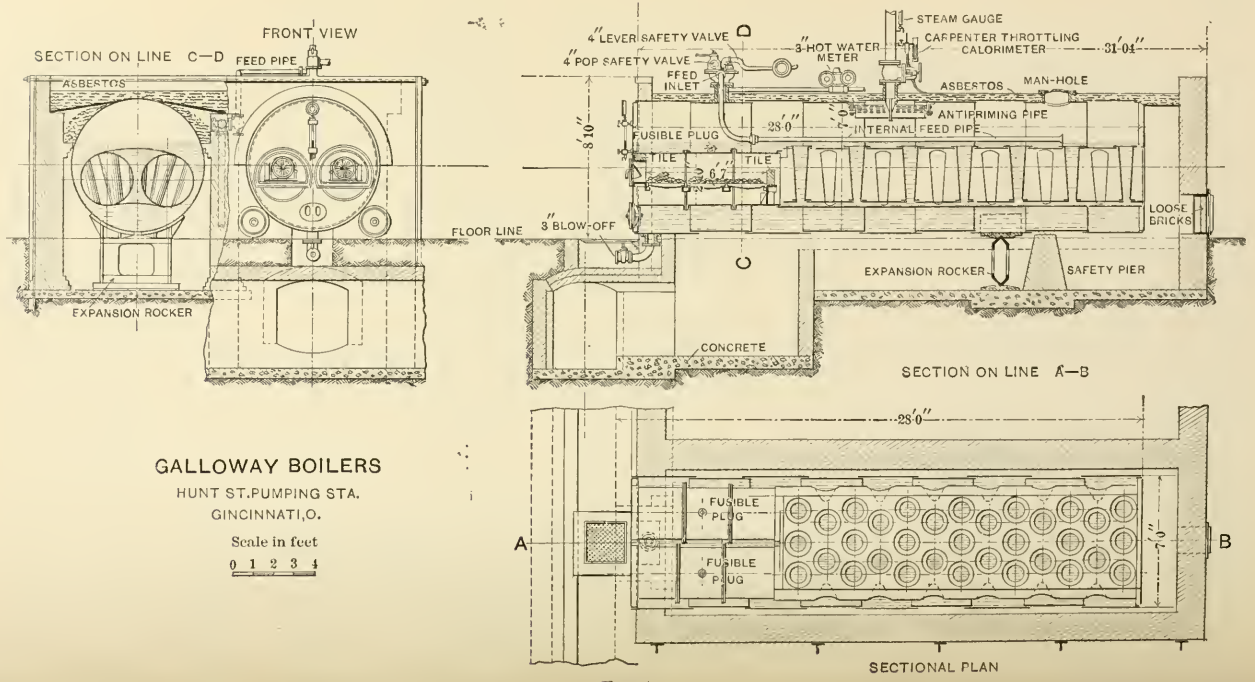

FRONT VIEW
SECTION ON LINE C—D
FEED PIPE
ASBESTOS
EXPANSION ROCKER
FLOOR LINE
3″ BLOW-OFF
4″ LEVER SAFETY VALVE
4″ POP SAFETY VALVE
FEED INLET
3″ HOT WATER METER
STEAM GAUGE
CARPENTER THROTTLING CALORIMETER
31′-04″
ASBESTOS
MAN-HOLE
ANTIPRIMING PIPE
INTERNAL FEED PIPE
28′-0″
8′10″
FUSIBLE PLUG
TILE
TILE
6′7″
LOOSE BRICKS
EXPANSION ROCKER
SAFETY PIER
CONCRETE
SECTION ON LINE A—B
28′0″
FUSIBLE PLUG
FUSIBLE PLUG
A
B
7′0″
SECTIONAL PLAN
GALLOWAY BOILERS
HUNT ST. PUMPING STA.
CINCINNATI, O.
Scale in feet
0 1 2 3 4

FIG. 1.

The two Galloway boilers were each of the following dimensions:

Shell, 7 ft. diameter, 28 ft. long.

Fire-boxes (2), 2 ft. 10 ins. inside diameter, 8 ft. long.

Grates (2), 2 ft. 9 ins. wide, 6 ft. 7 ins. long.

Combustion flue, 2 ft. $11\frac{3}{4}$ ins. vertical diameter, and 3 ft. 4 ins. horizontal diameter, 20 ft. long.

Galloway circulating tubes (33), 6 ins. diameter small end, $10\frac{1}{2}$ ins. diameter large end, 2 ft. $11\frac{3}{4}$ ins. long.

The boilers were set so as to expose about two-thirds of the shell as heating surface. The heating surface of one boiler is divided as follows:

Heating surface, fire-boxes (2)	83.982	sq. ft.
" " combustion flue	296.935	"
Heating surface, Galloway tubes (33)	212.338	sq. ft.
" Rear head	20.916	"
" Shell	381.170	"
" Total	995.341	"
" Two boilers	1 990.68	"
Cross-section, Two fire boxes	12.608	"
" Combustion flue	11.444	"
" Smoke tunnel	15.155	"
" Sheet-iron chimney	12.566	"
Grate surface (2)	36.206	"

Ratio, heating to grate surface.....27.50 to 1

Ratio, grate surface to flue vent..... 3.16 to 1

Water surface, one boiler (mean level) 6 ft. $1\frac{1}{2}$ ins. wide, 28 ft. long.

Chimney (iron), 48 ins. diameter, 70 ft. high.

The length of the smoke traverse from the front end of the grates to the top of the chimney was as follows:

Boiler No. 1	162.37 ft.
Boiler No. 2	153.21 "
Separation of boiler centers	9 ft. 2 in.

On October 11th, during Test No. 28, and on November 18th, during Test No. 35, tests were made to ascertain the time required by the hot gases to flow from the front end of the fire-boxes to the

top of the chimney. This was accomplished by allowing the coal on the grates to burn to incandescence, and then throwing into the fire-box a pint of liquid asphaltum and quickly closing the fire-door. From these tests, the mean velocities of the gases through the fire-boxes, flues and chimney were as follows:

Boiler No. 1 ..... 10.825 ft. per second.
Boiler No. 2 ..... 11.785 "

The Galloway boilers were the only ones in the water department which were not supplied with some kind of a fuel-saving device or some other appliance which might interfere with a uniform treatment of all coals, and also be of sufficient capacity to admit of using the coals in reasonably large quantities. These boilers, without urging, made dry steam at the rate of 10 625 lbs. per hour from feed water at a temperature of 190° Fahr., and with a steam pressure of 102 lbs.; equivalent to 326.90 Commercial H. P. by the Centennial Standard of boiler power rating.

The coal was weighed from the sample lot in iron barrows carrying from 1 250 to 1 600 lbs., depending upon the kind of coal, and was fired directly from the barrows to the boilers. The time each barrow was fired from was noted, and the unburned coal in the barrow at the end of the test was weighed back and deducted from the aggregate charge.

Provision was made, upon receiving a sample shipment of coal at the pumping station, to store it so that rain or snow should not wet it, and each sample was kept separate from the coal used for the routine work of the station. The tare of the iron barrows was taken each time one was run out of the boiler house empty for a new load of coal, and the net weights so obtained were the weights of coal actually fired from each barrow.

Previous to the beginning of each test, the boilers, with clean grates, were fired from the sample coal, the steam pressure was brought to 102 lbs. by gauge, the temperature of the feed water was raised to 190° Fahr. (with few exceptions), and the pumping engine was started at normal speed. This procedure was always observed, and occupied usually about two hours.

When the charges of coal on the grates, just before starting a measured test, were no longer able to maintain the steam pressure

under normal conditions of engine speed, temperature, and rate of supply of feed water, the signal for starting was given, and for 16 hrs. thereafter all notes necessary to make a physical test of the coals were taken half-hourly.

At the end of each test the last charges of coal on the grates were continued, as at the beginning, until the steam pressure, with the normal speed of the pumping engine, could no longer be maintained, when the test was stopped.

The grates were cleaned, usually, at regular intervals, and all ash, clinker and unburnt coal was weighed back dry. The firemen were constantly under espionage during the tests, and were required to be alert and attentive, without exceeding the care which at all times should prevail in firing coal under steam boilers.

The water was supplied to the boilers through a pair of 3-in. hot-water, Worthington meters, made for the purpose, which were rated before and after each series of eight tests, and occasionally on shorter test intervals. The meters were rated under the same conditions of temperature of water and rate of delivery as during the coal tests, the water discharged being weighed in a tight tank on a platform scale, and reduced to volume corresponding to the temperature of the water passing through the meter. The meter tests in each series were sufficiently numerous to avoid error in determining the value of the factor required to reduce the meter reading to the true amount of water delivered to the boilers for the previous series of coal tests.

In anticipation of some criticism on the use of water meters for the measurement of water to the boilers, the following explanation of the conditions which brought about their use in preference to tanks and scales is offered:

The pumping station where these coal tests were conducted is in daily service, although usually for a shorter length of time each day than the interval of preparation for and test of each sample of coal, and upon the odd days when no tests of coal were in progress, the machinery was operated in the usual service, and at the completion of each coal test, the house was left in such condition that the pumps could be operated on the following morning, taking steam, however, from one or two of three other sets of boilers in the house, the Galloway boilers being reserved exclusively for the coal tests.

The feed pumps and pipes and feed-water heater were common to

all boilers, but during the test of a sample of coal were devoted exclusively to the test boilers, and at all other times to such other boilers as may have been in service. This arrangement prevented the usual breaking of connections in the general system of feed-water pipes to the ordinary service boilers, and compelled a measurement of the water to the test boilers at a point in the line of feed-water pipes which would effectually avoid any gain or loss of water before it was delivered to the test boilers; and as at such point (on top of the boilers) the water had passed through the feed-water heater and been brought to near the boiling point. meters were regarded as the only convenient means for this purpose.

The feed pipes from the meters to the check valves of the Galloway boilers were exposed to view, and no loss at joints could have occurred without its being promptly noticed; and no connections were made with these pipes through which either gains or losses from other boilers might have occurred. Thus all water which passed through the meters was delivered without gain or loss into the test boilers.

Recognizing the difficulty of a satisfactory measurement by meter of the water delivered to the boilers, careful tests of the meters were made previous to the coal tests, and at frequent intervals during the first few tests, until it was discovered that the wear of the meters followed a nearly uniformly varying curve; and in determining the amount of water to be credited to any test, it was assumed that the wear of the meters was uniformly variable between any two successive tests of the meters, and the gross error was properly distributed between the several preceding tests.

In testing the meter for error of registry, 25 cu. ft. by meter was taken for each draft in the tank, and usually five or six drafts were made to correct for slight variation in rates of flow. The error was obtained by comparison of the actual weight of water drawn with the weight at the observed temperature of the water registered by the meter.

Previous to each test and at frequent intervals the screws in the top of the meter case were loosened to vent any air or vapor which might have formed in the meter, but, excepting upon starting, there was seldom any air to vent, while the temperature of the water (190° Fahr.), and the pressure under which the meters were worked (105 lbs.) precluded the formation of vapor from the hot water.

The coal tests were for comparative rather than absolute effects, and so long as the method of water measurement was alike for all samples, and not subject to vagaries or phenomena, it was considered sufficient for the purpose. Meter readings were taken and compared half-hourly during the whole test of each sample of coal.

The water levels in each boiler were read from scales attached to the glass tubes and graduated to eighths of an inch. Any discrepancy between the first and last water levels was corrected by adding or subtracting the proper volume of water reduced to weight due to the temperature of the steam and water in the boilers at the end of test. By careful management of the boiler feeder it was often possible to obtain exactly the same water level at the end of the test as at starting, and during the tests to hold the water levels quite close to the starting point.

The readings, as taken for the two boilers, indicated the depth of water above the brass nuts at the lower ends of the glass gauges. These nuts were 1.875 ins. below the crowns of the fire-boxes, and this amount substracted from the readings on the scales gives the depth of water over the fire-boxes during the respective trials. In the tables which follow, the water levels are stated as the mean depth in inches over the fire-boxes.

The steam gauges and thermometers were of good make, and were compared from time to time with standard test instruments, and all readings were corrected for errors of registry. The quality of the steam was determined by a pair of "Carpenter" throttling steam calorimeters, obtained from the Schäffer and Budenberg Company, New York. One of each of these was set in the steam outlet at the midlength of each boiler, the deductions were averaged from each instrument, and the mean result used in computing the water entrained and the net steam for each test.

Calorimetric observations of the steam were taken every half hour during each test, although it was early discovered that the condition of the steam was a function of the boiler and not of the coal, the quality being quite the same for good and bad coals, and such variations as occurred from time to time during any test, and from coal to coal during the several months of test, were attributed to variations in the rate of steam flow to the pumping engines, and to the opening of the fire-doors at the time of taking calorimeter readings. The calorimeters

and steam connections were heavily clothed with non-conducting materials to prevent loss of heat to the atmosphere, or gain of heat from the steam pipe to the condensing bulb of the instrument.

During these tests over 3 600 observations were made with the steam calorimeters, and these were reduced and carefully studied for influence of the pumping engines on the quality of the steam.

Efficiency of the Steam.

With reference to the quality of steam, this should depend upon the type and setting of boilers, the rate of evaporation per unit of heating surface, and the manner in which the steam is taken from the boilers. Super-heating surface will be required to produce dry or super-heated steam, and a lack of super-heating surface in the design or setting of boilers means steam of thermal value lower than saturation. The quality of steam may be slightly affected by the fuel and firing. Thus a low-grade fuel, or careless firing, with frequent opening of the fire-doors and the rush of cold air into the fire-boxes or furnaces, may lower the quality of the steam and increase the amount of water entrained. But with coals not varying greatly in value, and with uniform care in firing, the quality of the steam should be a function of the boilers, the rate at which they are worked and the manner of taking the steam from the boilers.

If the steam is delivered at a steady rate into the atmosphere, or to an engine, where there is no sensible variation in the flow for any given rate of evaporation, the quality should be a maximum. When the delivery of steam is irregular and subject to large and rapid variations of flow, the quality should be a minimum.

At the time these tests were projected, it was thought that the quality of the steam would be quite uniform for all tests, varying inversely with the rate of evaporation, and rising and falling slightly with the quality of the coals; but no consideration had been given to the influence of the engines on the quality of the steam.

The pumping station contains two pumping engines, each of the same nominal capacity, viz., 6 000 000 galls. per 24 hours, delivered against a net head of 310 ft. One of these is a plain compound condensing Worthington engine, and the other (the Niles engine), a pair of non-condensing western river steamboat style of engines, coupled, with cranks set at quarters, and steamboat poppet valves operated by

rocking toes and lifters. The operation of the steam valves in the Worthington engine is smooth, and without sensible shock to the column of steam in the steam pipe moving from the boilers to the engine, while the steam valves of the other engine are abruptly opened and closed, producing, while the valves are open, a high velocity in the steam pipe, and bringing the column of steam quickly to a state of rest when the poppet valves are closed. Both the opening and closing movements of the steam valves of the Niles engine are rapidly effected by quick-acting cams, wholly unlike the smooth easy motion of the slide valves in the duplex Worthington engine.

When the steam was taken from the boilers by the Worthington engine, the quality was high; and when taken by the Niles engine, the efficiency of the steam dropped from 1 to 1½ per cent. The fact had been observed for some time before the cause was suspected; but the uniformly higher quality of steam when the Worthington engine was in service demonstrated that the engine itself had an influence on the quality of the steam, and a probable influence on the economical effect of the coal.

Of the fifty-four tests, twenty-four were made with the Niles engine, three partly with the Worthington engine and partly with the Niles engine, and one trial was made partly with the Worthington engine, and partly by blowing the steam through the feed-water heater into the atmosphere. The other twenty-six tests were made with the Worthington engine.

With the exception of Test No. 50 (which was purposely made partly with each engine), changes from one engine to the other, or in the mode of disposing of the steam, were caused by accidents, which compelled the stopping of the damaged engine and the starting of the other; or, as in one instance, the wasting of the steam into the air for a few hours.

Test No. 1 was made for 11 hrs. 56 mins. with the steam going to the Worthington engine, and for 4 hrs. 14 mins. with the steam going through the feed-water heater to the atmosphere.

Test No. 2 was made for 5 hrs. 8 mins. with the steam going to the Worthington engine, and for 11 hrs. 3 mins. with the steam going to the Niles engine.

Tests Nos. 3 to 19, inclusive, were made with the Niles engine.

Test No. 20 was made for 7 hrs. 13 mins. with the Niles engine, and for 8 hrs. 47 mins. with the Worthington engine.

Tests Nos. 21 to 41, inclusive, were made with the Worthington engine.

Tests Nos. 42 to 49, inclusive, were made with the Niles engine.

Test No. 50 was made for 10 hrs. with the Worthington engine, and for 5 hrs. 41 mins. with the Niles engine.

Tests Nos. 51 to 55, inclusive, were made with the Worthington engine.

Tests Nos. 2, 20 and 50, having been made partly with the Niles engine and partly with the Worthington engine, an opportunity was offered to inquire into the influence (if any) of the engines on the efficiency of the coal.

TABLE No. 1.—Comparison of Effects with Worthington and Niles Engines, Respectively, in Service.

Number of test.	Engine in service.	Pounds of coal per cubic foot of water supplied to boilers.
2	Worthington.	6.549
2	Niles.	7.056
20	Worthington.	7.132
20	Niles.	7.132
50	Worthington.	6.682
50	Niles.	6.696

During the early part of Tests Nos. 2 and 50, the Worthington engine was in service, and during the latter part the Niles engine. During the early part of Test No. 20 the Niles engine was in service, and during the latter part the Worthington engine. Considering these tests, there is a distinct, though rough, indication that the engine through which the steam was worked had an influence on the economy of the several fuels. Comparisons of steam, water and coal supplied to the boilers during intervals of the tests when both engines were (at times) in service, show, in all instances but one, that the coal per unit of water was greatest when the Niles engine was running, while in the single exception the coal per unit of water was almost precisely the same for each engine.

The notes from the calorimeters uniformly show a lower efficiency when the steam was going to the Niles engine; and for a while it was thought that the loss in steam efficiency was probably compensated by

a corresponding increase in the delivery of water to the boilers: but a careful review and comparison of the coal and water supplied to the boilers when both engines—at different times—were in service, show that not only was the quality of the steam lower, but the coal required

TABLE No. 2.—Efficiencies of the Steam.

1 No. of test.	2 Coal under test.	3 Worthington engine and to atmosphere.	4 Niles engine.	5 Worthington engine.	6 Worthington and Niles engines.
1....	New River. Mine Run	0.9898			
2....	Kanawha. Steam Lump				0.9873
3....	Kanawha. Gas Lump		0.9874		
5....	Pittsburg. Catsburg. Lump		0.9880		
6....	" " Barge Run		0.9836		
7....	" " Mine Run		0.9825		
8....	" " Nut and Slack		0.9788		
9....	" Woods. Lump		0.9824		
10....	" " Barge Run		0.9794		(Mean of
11....	" " Mine Run		0.9821		Tests Nos.
12....	" " Nut and Slack		0.9818		3 to 19.
13....	" Brown. Lump		0.9845		0.9821.)
14....	" " Mine Run		0.9854		
15....	" " Barge Run		0.9803		
16....	" " Nut and Slack		0.9823		
17....	" Cin'ti. Mine Run		0.9797		
18....	" " Nut and Slack		0.9782		
19....	Campbell's Creek. Nut and Slack		0.9770		
20....	Loup Creek. Mine Run				0.9851
21....	" " Nut and Slack			0.9901	
22....	Pocahontas. Mine Run			0.9902	
23....	Winifrede. Lump			0.9903	
24....	" Mine Run			0.9906	
25....	" Nut and Slack			0.9902	
26....	Pocahontas. Lump			0.9900	
27....	Loup Creek. Lump			0.9929	
28....	Thacker. Mine Run			0.9956	
29....	Campbell's Creek. Mine Run			0.9956	(Mean of
30....	Thacker. Nut and Slack			0.9949	Tests Nos.
31....	New River. Mine Run			0.9935	21 to 41.
32....	Acme. Mine Run			0.9930	0.9930.)
33....	" Nut and Slack			0.9913	
34....	Keystone. Mine Run			0.9939	
35....	New River. Lump			0.9933	
36....	Monongah. Mine Run			0.9935	
37....	Thacker. Lump			0.9937	
38....	Belmont Splint. Mine Run			0.9949	
39....	" " Nut and Slack			0.9945	
40....	Jellico. Ky.. Nut and Slack			0.9913	
41....	Luhrig. Washed Egg			0.9949	
42....	Logan Cons.. Thacker. Mine Run		0.9851		
43....	Luhrig. Washed Slack		0.9895		
44....	Wellston Shaft. Mine Run		0.9805		(Mean of
45....	" " Nut and Slack		0.9769		Tests Nos.
46....	Maritime. Thacker. Mine Run		0.9767		42 to 49.
47....	Jellico. Ky.. Mine Run		0.9798		0.9797.)
48....	Jellico. Tenn.. Nut and Slack		0.9784		
49....	New River. Lump		0.9771		
50....	Thacker. Mine Run				0.9834
51....	Eagle. Mine Run				
52....	Cedar Grove. Mine Run				
53....	Mt. Carbon, Mine Run			0.9930	
54....	Chesapeake. Mine Run				
55....	Coalburg. Mine Run				

per unit of water supplied to the boilers was higher with the Niles engine.

Table No. 2 shows the efficiencies of steam for Tests Nos. 1 to 50, inclusive. In Column 3 is given the mean efficiency of the steam when it was consumed for the major portion of the test by the Worthington engine, and blown through the feed-water heater into the air for the latter portion of the test. Column 4 shows the efficiencies of the steam when the Niles engine alone was working. Column 5 contains the same data when the Worthington engine alone was working; and Column 6 the efficiencies when the steam was taken for a portion of the test by the Niles engine, and for a portion of the test by the Worthington engine.

For Tests Nos. 51 to 55, inclusive (Worthington engine), the calorimeters were read regularly as in all previous tests, but, the notes simply confirming the results obtained from Tests Nos. 21 to 41, inclusive, the mean efficiency (0.993) for these former tests was used in determining the weight of water actually converted into steam for the last five coals.

It was unfortunate that all coals could not have been tested with the same engine taking the steam, but accidents, first to one engine and then to the other, made this an impossibility, and recourse must be had to such corrections for the engines as the data will warrant.

It will be understood that, throughout the tests mentioned, when first one engine and then the other was used to work off the steam made by the test boilers, the same coal was burned, and when comparisons are made of the influence of the engines on the coal required per unit of water, coal always of the same commercial quality is under consideration.

Considering the New River coals, which were tested first (Test No. 1), by working the steam for nearly 12 hours through the Worthington engine, and then for over 4 hours through the feed-water heater to the atmosphere; next (Tests Nos. 31 and 35), when the Worthington engine used the steam for the whole of each test; and finally (Test No. 49), when the Niles engine used the steam for the entire trial, it appears that the efficiency of the steam varied perceptibly under these different conditions of use, and that while the boilers, firemen and fuel were not changed, the change in the manner of using or wasting the steam had an influence on its quality.

TABLE No. 3.—Efficiency of Steam with New River Coals.

Test No.	Kind of coal.	Steam pressure. In pounds.	Disposition of steam.	Efficiency, saturated steam being taken as unity.
1....	New River, Mine Run..	100.5	To Worthington engine, 11 hrs. 56 mins.; to feed-water heater, 4 hrs. 14 mins.....	0.9898
31....	New River, Mine Run..	101.4	To Worthington engine.....	0.9935
35...	New River, Lump.....	101.3	To Worthington engine	0.9933
49....	New River, Lump.....	100.9	To Niles engine.....	0.9771

By this comparison it appears that the efficiency of the steam was 1.63% higher when the Worthington engine was used than when the Niles engine was used, and to avoid prejudice to any coal investigated during these tests, the author concluded that a careful study of all the conditions and data warranted the assumption that the extreme variations in the efficiency of the steam were not due either to the kind or grade of coal burned, but to the manner of using the steam from the boilers, and that for purposes of comparison the coals should be rated upon a uniform efficiency of the steam.

Condition of the Boilers.

The tests were made with the boilers in ordinary condition, and at the end of each series of three or four tests, the boilers were opened and all loose material washed out. The feed water was passed through an Excelsior filter and feed-water heater, which intercepted the mud and heavier suspended matter before it reached the boilers. At each periodical washing a small amount of loose scale and fine silt was removed from the circulating tubes and the bottom of the boilers. After Test No. 49 the boilers were opened and the small amount of scale found was carefully chipped and scraped off to the surface of the metal. Test No. 50 was then made, using a coal the value of which had been previously established during Test No. 28 with the boilers in ordinary condition.

Assuming that Test No. 28 represented the boilers in the condition for all coals tested, while Test No. 50 represented the boilers in the best possible condition, Table No. 4 shows the comparison of effects for these two tests.

TABLE No. 4.—Comparison of Steam per Pound of Coal and per Pound of Combustible for Tests Nos. 28 and 50 (in both using Thacker, Mine-Run Coal).

Number of Test.	28	50
Duration of test.	16 hrs. 00 mins.	15 hrs. 41 mins.
Steam per pound of coal from and at 212° Fahr. Pounds	9.392	9.735
Steam per pound of combustible from and at 212° Fahr. Pounds	10.132	10.357
Percentage of combustible	92.707	93.993
Increased percentage of combustible (second trial of coal)	.....	1.387
Apparent gain in economy by coal. Percentage	.....	3.650
Net gain in economy by coal. Percentage	.....	2.263
Gain in economy by combustible. Percentage	.....	2.220

From this comparison it appears that the difference between the performance of a given coal with the boilers very carefully cleaned, and with the boilers in ordinary working condition, was about 2¼%, an amount by which the economic and capacity results obtained with each coal may be increased to show the possibilities of these several coals with perfectly clean heating surfaces and ordinary firing.

Sources of Coal Samples.

Fifty-four samples of coal were tested, as follows:

Fourteen samples of Pennsylvania (Pittsburg) coals.

Thirty-three samples of West Virginia coals.

Four samples of Ohio (Hocking Valley) coals.

Two samples of Kentucky coals.

One sample of Tennessee coal.

The Pennsylvania coals were as follows:

Four samples from the Brown Mines, Nos. 10 and 11, Second Pool, Monongahela River.

Four samples from the Catsburg Mine, No. 47, Third Pool, Monongahela River.

Two samples from the Cincinnati Mines, Third Pool, Monongahela River.

Four samples from the Woods Mine, No. 75, Fourth Pool, Monongahela River.

The West Virginia coals were as follows:

Four samples from New River, Mines Nos. 71 to 80, inclusive, Fayette Co.

Three samples from Loup Creek, Mines Nos. 81 to 87, inclusive, Fayette Co.

Two samples from Pocahontas, Mines Nos. 96 to 140, inclusive, McDowell and Mercer counties.

One sample, Kanawha steam coal from Mine No. 47, Kanawha Co.

One sample, Kanawha gas coal from Mine No. 102, Kanawha Co.

Two samples, Acme coal from Mine No. 24, Kanawha Co.*

One sample, Keystone coal from Mine No. 23, Kanawha Co.*

Three samples, Winifrede coal from Mines Nos. 14, 17, 18 and 19, Kanawha Co.

Four samples, Thacker coal, Mingo Co.

One sample, Logan Consolidated Thacker coal, Mingo Co.

One sample, Maritime Thacker coal, Mingo Co.

Two samples, Campbell's Creek coal from Mines Nos. 10 to 12, inclusive, Kanawha Co.

One sample, Monongah coal from Mines Nos. 2 to 5, inclusive, Marion Co.

One sample, Eagle coal from Mine No. 5, Kanawha Co.

One sample, Cedar Grove coal from Mine No. 27, Kanawha Co.

One sample, Mt. Carbon coal from Mines Nos. 48 and 49, Fayette Co.

One sample, Chesapeake coal from Mine No. 36, Kanawha Co.

One sample, Coalburg coal from Mine No. 21, Kanawha Co.

Two samples, Belmont Splint coal from Mine No. 32, Kanawha Co.

The Ohio coals were as follows:

Two samples, Luhrig coal from Mine No. 1 (Daleton), Athens Co.

Two samples, Wellston Shaft coal from Mine No. 3, Jackson Co.

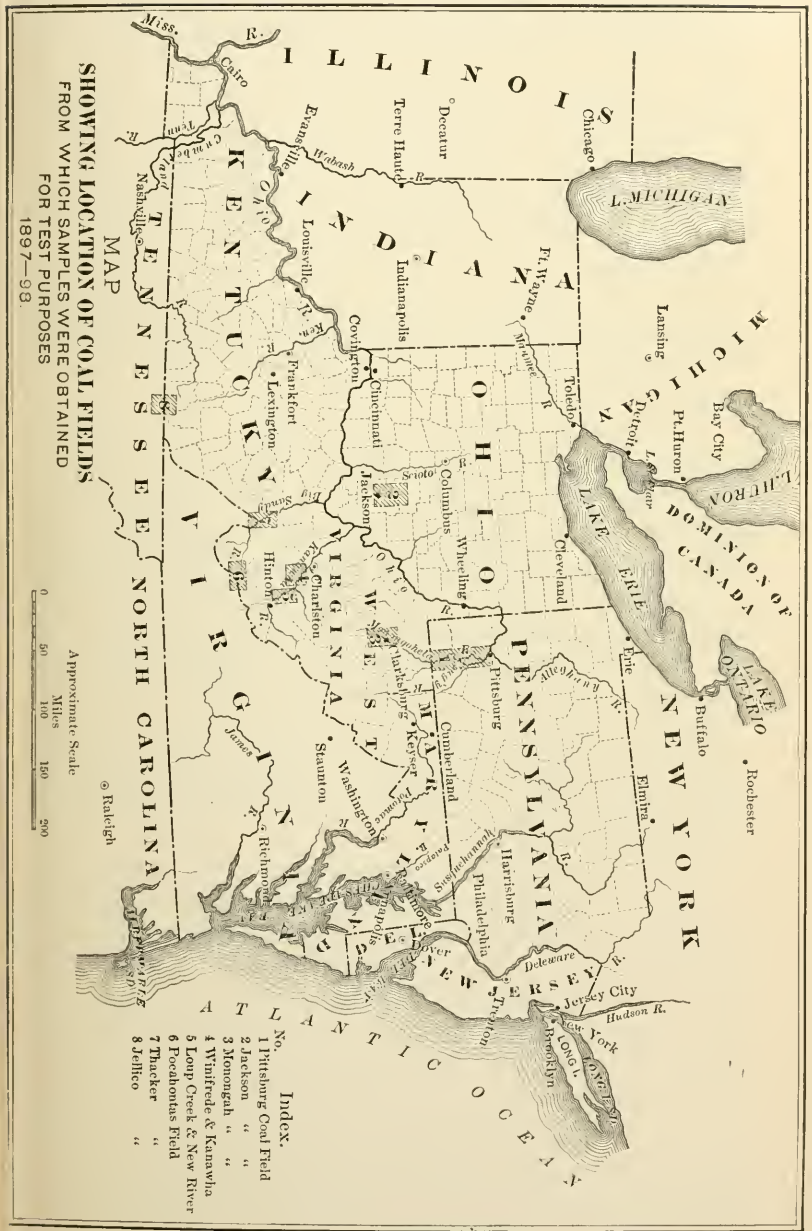
The Kentucky coals were as follows:

Two samples from the Jellico Mines in Whitely Co.

The single sample of Tennessee coal was from the Jellico Mines, in Campbell Co.

The sketch map (Fig. 2) indicates the geographical location of the various coal fields from which the test samples of coal were obtained.

* These coals represent two separate and distinct veins.


MAP
SHOWING LOCATION OF COAL FIELDS
FROM WHICH SAMPLES WERE OBTAINED
FOR TEST PURPOSES
1897—98
Index.
No.
1 Pittsburg Coal Field
2 Jackson " "
3 Monongah " "
4 Winifrede & Kanawha
5 Loup Creek & New River
6 Pocahontas Field
7 Thacker "
8 Jellico "
Approximate Scale
Miles
0 50 100 150 200
ILLINOIS
INDIANA
OHIO
KENTUCKY
TENNESSEE
WEST VIRGINIA
PENNSYLVANIA
NEW YORK
MICHIGAN
VIRGINIA
NORTH CAROLINA
MARYLAND
NEW JERSEY
DOMINION OF CANADA
L. MICHIGAN
LAKE ERIE
LAKE ONTARIO
ATLANTIC OCEAN
Chicago
Decatur
Terre Haute
Indianapolis
Evansville
Louisville
Cincinnati
Columbus
Cleveland
Toledo
Detroit
Lansing
Bay City
Pt. Huron
Buffalo
Rochester
Erie
Elmira
Pittsburg
Harrisburg
Philadelphia
Jersey City
New York
Brooklyn
Long I.
Cumberland
Wheeling
Charleston
Kanawha
Hinton
Staunton
Richmond
Norfolk
Lexington
Frankfort
Nashville
Cairo
Miss. R.
Ohio R.
Wabash R.
Hudson R.
Delaware R.
Susquehanna R.
Raleigh

In each instance the samples of coal were ordered from the agents or mine owers in quantity sufficient for the preliminary firing of the boilers and for the 16-hour test which followed, and each agent or mine owner was required to furnish with the sample of coal a sworn statement of the source and market grade of coal supplied. While this may not in all cases have prevented the substitution of coal from one mine for that of another, it put the coal men on their honor to supply the kind and grade of coal ordered by the city for test purposes.

The importance of these tests to Cincinnati will appear when it is known that the City Government alone is a purchaser of over $80 000 worth of coal annually, and the value of the entire consumption of coal for all purposes in the city at the present time cannot be less than $2 500 000 per annum.

The most extensive series of physical tests of coal under uniform conditions, previous to these, of which the author is aware, were made for the United States Navy by Professor W. R. Johnson, in 1842 and 1843. These comprised physical and chemical examinations, and calorific determinations by the Berthier method, of forty-four different samples of coal. In view of the very large quantities of coal burned and water evaporated during each of the author's trials, it is amusing to read from Professor Johnson's report, his description of the magnitude of these earlier (physical) trials for the Navy:

"The total weight of coal consumed in the trials of evaporative power has been nearly 62.5 tons, and the weight used on an average 978 lbs. per trial. This statement may be sufficient to indicate that the experiments have been made on a scale unobjectionable on the score of magnitude."

Presuming that few members of the Society have any knowledge of these earlier coal tests by Professor Johnson, it will be interesting to such to note that the experiments were conducted with a single return-flue boiler, 3 ft. 6 ins. in diameter and 30 ft. long, with two return flues the full length of the boiler, each 10 ins. in diameter. The air to support combustion was heated before it passed under the grate, in flues built in the side walls of the boiler setting, and the gases of combustion passed from the grate backward under the shell of the boiler, thence through the flues to the front end of the boiler, thence to the rear end of the boiler through a flue in the brickwork on the left side above the fire line, and thence forward through a similar flue in the right side of the setting, to the base of the brick chimney, set

at the front end and to the right of the boiler. The chimney was of brick 41 ft. high from ground level, with a flue 18 ins. square, and a sheet iron extension 22 ft. high and 23 ins. in diameter. This boiler was worked at an average of about 6.25 lbs. per square inch above atmospheric pressure—the pressure never being less than 5.8 lbs.—and upon two occasions only, during this long series of trials, the pressure was raised to 15.65 and 19.16 lbs., respectively. All steam was blown through two direct, weighted, safety valves into the atmosphere or into the chimney.

Considering the low steam pressures used, and the moderate rate of evaporation per unit of heating surface, the capacity of this old-style boiler during the coal tests was about 30 Commercial H. P.

Physical Tests of Coals.

In the following tables giving the *résumé* of each test, the coals have been grouped by States, and the several grades of each kind of coal tested are brought together for comparison of economic results. The favorite grades of steam coals are Mine Run and Nut and Slack, with an extensive use of Slack coal for mechanical stokers. The prices of the coals grade downward from Screened Lump to Slack, and it is constantly a problem with buyers of large quantities of coal to determine which grade at given prices will represent the best economy. Each grade of coal has its advocates among buyers. In spite of the millions of tons of graded coals which have been burned on western river steamboats, it is still a matter of controversy among river men whether Mine Run or Screened Lump coal, at the prices usually charged, is the cheaper coal for steam purposes.

It is customary when burning low-grade bituminous coals to wet the coal slightly before it is charged into the furnace. With many of such coals, this preliminary wetting causes the smaller particles to adhere together until sufficiently heated to make a coke, and prevents the usual sifting of these particles through the grates into the ash-pits with corresponding loss of useful effect; and if the coal be not too dirty, it is often possible by this method to obtain quite as economical results with Slack as with Nut and Slack coal. Owing to the difficulty of determining just how much water each particular kind of coal might take without positive injury to its efficiency, during these tests, the low-grade coals were always fired without pre-

liminary wetting, and from the dealers' point of view, the coal should be valued without regard to subsequent treatment by the buyer. The coals, therefore, in all cases were burned in the same condition as supplied to the station by the dealers or mine owners.

Previous to each trial, 100 lbs. of coal was taken from the sample, and put into a wooden box perforated on the top, bottom and sides, to allow free circulation of air around and through the coal, and placed on one of the steam manheads of the boilers to dry the coal and obtain a factor for estimation of the superficial moisture it contained. At the end of the test this box and sample was returned to the scale, and any loss of weight was taken as the excess of moisture in the coal. The losses estimated in this manner ranged from 0 to 2%, and have been allowed for in the item "Commercially dry coal burned."

The results given in the tables are in no case supposed to indicate the best possibilities with these coals. The firemen were perhaps the best regularly employed by the City Water Department, and carefully and faithfully performed their duties during these tests, but no attempt at expert firing was permitted, the men being directed only to give uniformly good service throughout each and every test. No extra help was furnished for handling the coal, and as a whole, the results are no better than good firemen should obtain in steady service from day to day, with the same coals. The author regards the results obtained by this method of handling the coal as more satisfactory to the buyers and users of coal than those which might have been obtained by expert firemen with assistants to open and close the fire doors, and an amount of labor which men could not maintain day after day.

Referring to Tables Nos. 5 to 11, inclusive, when the "duration of test" (1) as in six instances, was a few minutes more or less than 16 hours, this was occasioned by the steam pressure or water levels in the boilers being too high or too low, or by the condition of the charges of coal on the grates, which compelled an earlier or later stopping of the respective tests. In each test an attempt was made to restore the starting conditions of steam pressures, water levels, fires on grates and speed of pumping engine at the expiration of 16 hours of test, and with the exceptions noted, this was always accomplished. The steam gauges (2), and thermometers (6), (7), (9), were compared for each test with

standard instruments. The temperature of the waste gases in the chimney (4) was taken with a Brown hot-blast pyrometer, which was adjusted before each test by a standard mercury thermometer.

"Commercially dry coal" (10) means the actual weight of coal fired to the boilers, reduced by the amount of moisture lost by a sample of 100 lbs. carried for 16 hours in a perforated box on top of the boilers. This does not represent the moisture found by chemical analysis of the coal samples, but the wetness due to exposure to the weather in barges and railway cars. During the hot, dry weather of August and September, 1897, but few of the coals showed any loss when placed in the box over the boilers. The conditions with reference to determining the percentage or amount of dry coal were alike for all coals. The ash, clinker and refuse from the ash-pits (11) in all instances was weighed back dry.

Coal which fell into the ashes from the firemen's shovels was picked out and burned, and, except during the first three or four tests, but few pieces of unburned or partially coked coal were found among the ashes.

The Wellston shaft coal, used during Tests Nos. 44 and 45, made a very light feathery ash, much of which was carried over the bridge walls by the draught through the fire boxes while the fires were being broken and cleaned.

The "water supplied to the boilers" (13) represents the water by meter, corrected for error, and reduced to weight for observed temperature of feed water (9). The "efficiency of the steam" (15) is obtained by dividing the average thermal value of the steam by calorimeter, by the total heat of saturated steam at the observed pressure. This takes into account the heat in the entrained water as well as that in the saturated steam, and credits the coal with the whole work done by the boilers; and since these tests were for relative coal effects, and not boiler effects, it seemed to the author entirely proper to credit each coal with the whole heat accounted for by the steam calorimeters.

The "net water converted into steam" (16) is the "water supplied to boilers" (13) multiplied by the "efficiency of the steam" (15). As the efficiency recognizes a small amount of heat in the "water entrained in the steam" (14), the term "net water converted into steam" is therefore not strictly correct.

In reducing "steam per pound of coal" (17) or "combustible"

(19) "from temperature of feed," to "steam per pound of coal" (18), or "combustible" (20), "from and at 212° Fahr.," the divisor 996 instead of 965.7 was used. The remaining captions will require no explanation.

While the physical results from the different coals are properly comparable with each other, they are not comparable with the results obtained from the same coals under other conditions of boilers, furnace, firemen, etc. (This remark, of course, does not apply to the proximate chemical analyses and determinations of thermal value, which will be referred to hereafter.)

The evaporative results from and at 212° Fahr. of Test No. 50 (Table No. 9), and Tests Nos. 51 to 55, inclusive (Table No. 10), have been corrected to agree with the data of Table No. 4, to correspond with the first forty-nine tests, all of which were made prior to the careful scaling of the boilers.

TABLE No. 5.—Pennsylvania Coals.

	Kind of coal	Pittsburg—Brown, Second Pool.				Pittsburg—Catsburg, Third Pool.			
	Grade of coal	Lump.	Barge Run.	Mine Run.	Nut & Slack	Lump.	Barge Run.	Mine Run.	Nut & Slack
	Number of test	13	15	14	16	5	6	7	8
	Date of test	Sept. 1, '97.	Sept. 8, '97.	Sept. 3, '97.	Sept. 10, '97.	Aug. 9, '97.	Aug. 11, '97.	Aug. 13, '97.	Aug. 16, '97.
1	Duration of test	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.
2	Av. steam pressure, lbs	101.797	101.594	101.820	101.203	103.109	101.500	101.656	101.023
3	" depth of water over fire boxes, ins	5.582	5.767	5.541	5.480	6.906	6.983	6.672	6.211
4	" temp. waste gases, Fahr	645.156°	603.950°	647.047°	609.470°	675.010°	736.150°	733.975°	707.930
5	" draft, gauge in chimney, ins	0.457	0.425	0.454	0.460	0.371	0.444	0.411	0.422
6	" temp. of air during test, Fahr	80.141°	81.187°	73.953°	82.040°	80.360°	75.580°	77.281°	72.094°
7	" dryness of air " " Fahr	7.657°	11.312°	9.984°	12.310°	9.340°	9.635°	12.336°	8.406°
8	" barometer " " ins	29.534	29.666	29.732	29.711	29.400	29.360	29.548	29.436
9	" temp. of feed water, Fahr	196.828°	196.856°	196.220°	194.780°	194.400°	193.547°	194.105°	196.210°
10	Commercially dry coal burned, lbs	20 166.00	20 037.00	21 413.00	21 623.00	19 441.00	20 805.00	21 345.00	21 823.00
11	Ash and clinker, weighed back dry, lbs	1 125.00	1 200.00	1 914.00	1 878.00	1 262.00	1 469.00	1 922.00	2 186.00
12	Percentage of non-combustible, %	5.579	5.989	8.937	8.685	6.491	7.061	9.004	10.017
13	Water supplied to boilers during test, lbs	173 117.301	173 150.484	171 861.891	173 637.407	170 561.758	173 805.131	172 922.832	171 080.318
14	Water entrained in the steam, corrected %	0.965	0.965	0.965	0.965	0.965	0.965	0.965	0.965
15	Efficiency of the steam, corrected	0.993	0.993	0.993	0.993	0.993	0.993	0.993	0.993
16	Net water converted into steam, lbs	171 905.480	171 938.431	170 658.858	172 421.915	169 367.826	172 588.495	171 712.372	169 882.756
17	Steam per lb. of coal from temp. of feed, lbs	8.524	8.581	7.970	7.974	8.712	8.295	8.045	7.785
18	Steam per lb. of coal from and at 212° Fahr., lbs	9.004	9.064	8.424	8.439	9.227	8.790	8.521	8.227
19	Steam per lb. of combustible from temp. of feed, lbs	9.028	9.128	8.752	8.732	9.317	8.926	8.841	8.651
20	Steam per lb. of combustible from and at 212° Fahr., lbs	9.537	9.642	9.251	9.241	9.868	9.459	9.364	9.143
21	Steam per sq. foot of heating surface, per hour, lbs	5.397	5.398	5.358	5.413	5.317	5.419	5.391	5.334
22	Coal per sq. foot of grate surface, per hour, lbs	17.408	17.297	18.484	18.666	16.783	17.960	18.426	18.839
23	Relative efficiency on coal	0.881	0.887	0.825	0.826	0.903	0.860	0.834	0.805
24	Relative efficiency on combustible	0.885	0.895	0.859	0.858	0.916	0.878	0.869	0.849

TABLE No. 6.—Pennsylvania Coal.

		Pittsburg—Cincinnati Third Pool.		Pittsburg—Woods, Fourth Pool.			
	Kind of coal						
	Grade of coal	Mine Run.	Nut & Slack	Lump.	Barge Run	Mine Run.	Nut & Slack
	Number of test	17	18	9	10	11	12
	Date of test	Sept. 14, '97.	Sept. 16, '97.	Aug. 20, '97.	Aug. 23, '97.	Aug. 25, '97.	Aug. 27, '97.
1	Duration of test	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs. 6 m.	16 hrs.
2	Av. steam pressure, lbs.	101.047	100.875	100.531	100.750	101.266	100.227
3	" depth of water over fire boxes, ins.	5.607	5.717	5.779	5.876	5.562	5.234
4	" temp. of waste gases, Fahr.	654.840°	631.656°	682.500°	679.897°	656.000°	661.453°
5	" draft, gauge in chimney, ins.	0.444	0.455	0.427	0.450	0.451	0.444
6	" temp. of air during test, Fahr.	83.508°	84.922°	71.797°	70.164°	70.141°	78.062°
7	" dryness of air " " Fahr.	10.742°	10.281°	9.547°	5.188°	7.313°	9.046°
8	" barometer " " ins.	29.657	29.572	29.584	29.469	29.444	29.448
9	" temp. of feed water, Fahr.	194.852°	196.150°	196.406°	195.206°	195.981°	196.960°
10	Commercially dry coal burned, lbs.	20 839.00	20 922.00	20 707.00	21 352.00	20 508.00	21 393.00
11	Ash and clinker weighed back, dry, lbs.	1 272.00	1 503.00	1 961.00	2 057.00	2 010.00	2 210.00
12	Percentage of non-combustible, %	6.103	7.183	9.470	9.634	9.801	10.330
13	Water supplied to boilers during test, lbs.	171 998.332	172 904.860	172 403.533	173 151.440	174 518.825	172 453.538
14	Water entrained in the steam, corrected, %	0.965	0.965	0.965	0.965	0.965	0.965
15	Efficiency of the steam, corrected	0.993	0.993	0.993	0.993	0.993	0.993
16	Net water converted into steam, lbs.	170 794.344	171 694.526	171 196.708	171 939.380	173 297.193	171 246.363
17	Steam per lb. of coal from temp. of feed, lbs.	8.196	8.206	8.268	8.053	8.450	8.005
18	Steam per lb. of coal from and at 212° Fahr., lbs.	8.673	8.672	8.735	8.519	8.933	8.452
19	Steam per lb. of combustible from temp. of feed, lbs.	8.729	8.841	9.132	8.911	9.368	8.927
20	Steam per lb. of combustible from temp. and at 212° Fahr., lbs.	9.237	9.344	9.648	9.426	9.903	9.426
21	Steam per sq. foot of heating surface, per hour, lbs.	5.362	5.391	5.375	5.398	5.407	5.377
22	Coal per sq. foot of grate surface, per hour, lbs.	17.990	18.061	17.875	18.432	17 593	18.468
23	Relative efficiency on coal	0.849	0.849	0.855	0.834	0.874	0.827
24	Relative efficiency on combustible	0.858	0.867	0.896	0.875	0.919	0.875

TABLE No. 7.—West Virginia Coals.

Kind of coal	New River.				Loup Creek.			Pocahontas.	
Grade of coal	Lump.	Lump.	Mine Run.	Mine Run.	Lump.	Mine Run.	Slack.	Lump	Mine Run.
Number of test	35	49	1	31	27	20	21	26	22
Date of test	Nov. 18, '97.	Dec. 21, '97.	July 27, '97.	Nov. 8, '97.	Oct. 7, '97.	Sept. 20, '97.	Sept. 22, '97.	Oct. 5, '97.	Sept. 24, '97.
1 Duration of test	16 hrs.	16 hrs.	16 hrs. 10 m.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.
2 Av. steam pressure, lbs	101.336	100.906	100.485	101.387	101.566	102.289	98.289	101.402	101.500
3 Av. depth of water over fire boxes, ins	5.671	5.599	7.503	5.662	5.498	5.728	5.679	5.548	5.516
4 Av. temp. waste gases, Fahr	622.656°	606.484°	631.300	521.720°	637.562°	718.812°	637.828°	625.969°	632.719°
5 " draft, gauge in chimney, ins	0.496	0.486	0.375	0.421	0.468	0.527	0.460	0.443	0.460
6 " temp. of air during test, Fahr	42.027°	35.062	84.250°	59.797°	60.687°	57.375°	62.484	71.617°	68.922°
7 " dryness of air " " Fahr	4.930°	2.281°	9.110°	1.555°	8.206°	9.125°	9.765°	10.865°	11.250°
8 " barometer " " ins	29.935	29.487	29.400	29.153	29.582	29.806	29.621	29.523	29.620
9 " temp. of feed water, Fahr	192.172°	195.005°	191.210°	184.918°	196.222°	195.380°	195.536°	194.697°	194.940°
10 Commercially dry coal burned, lbs	19 865.00	18 515.00	18 486.00	19 474.00	18 346.00	20 907.00	20 010.00	18 123.00	18 821.00
11 Ash and clinker, weighed back dry, lbs	1 394.00	795.00	1 608.00	841.00	719.00	851.00	955.00	1 102.00	1 318.00
12 Percentage of non-combustible, %	7.017	4.293	8.698	4.318	3.919	4.070	4.773	6.081	7.002
13 Water supplied to boilers during test, lbs	173 195.866	171 060.585	172 400.556	175 687.419	178 571.394	174 306.660	167 319.175	171 706.125	168 771.676
14 Water entrained in the steam, corrected %	0.965	0.965	0.965	0.965	0.965	0.965	0.965	0.965	0.965
15 Efficiency of the steam, corrected	0.993	0.993	0.993	0.993	0.993	0.993	0.993	0.993	0.993
16 Net water converted into steam, lbs	171 983.495	169 863.161	171 193.752	174 457.607	177 321.394	173 086.513	166 147.941	170 504.182	167 590.274
17 Steam per lb. of coal from temp. of feed, lbs	8.657	9.174	9.261	8.958	9.665	8.279	8.303	9.408	8.904
18 Steam per lb. of coal from and at 212° Fahr, lbs	9.186	9.707	9.834	9.572	10.215	8.759	8.776	9.958	9.423
19 Steam per lb of combustible from temp. of feed, lbs	9.310	9.586	10.143	9.363	10.060	8.630	8.719	10.017	9.575
20 Steam per lb of combustible from and at 212° Fahr., lbs	9.879	10.143	10.771	10.005	10.633	9.130	9.216	10.603	10.133
21 Steam per sq. foot of heating surface, per hour, lbs	5.400	5.333	5.319	5.477	5.567	5.434	5.216	5.353	5.262
22 Coal per sq. foot of grate surface, per hour, lbs	17.149	15.590	15.794	16.811	15.837	18.048	17.272	15.645	16.247
23 Relative efficiency on coal	0.899	0.950	0.963	0.937	1.000	0.857	0.859	0.975	0.922
24 Relative efficiency on combustible	0.917	0.942	1.000	0.929	0.987	0.848	0.856	0.984	0.941

TABLE No. 8.—West Virginia Coals.

Kind of coal	Kanawha Steam.	Kanawha Gas.	Acme.		Keystone.	Winifrede.		
Grade of coal	Lump.	Lump.	Mine Run.	Nut & Slack	Mine Run.	Lump.	Mine Run.	Nut & Slack
Number of test	2	3	32	33	34	23	24	25
Date of test	July 29, '97.	July 31, '97.	Nov. 10, '97.	Nov. 12, '97.	Nov. 15, '97.	Sept. 27,'97.	Sept. 29,'97	Oct. 1, '97.
1 Duration of test	16 hrs. 11 m.	16 hrs. ½ m.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.
2 Av. steam pressure, lbs.	102.934	102.370	101.812	102.195	102.227	102.387	102.262	101.809
3 " depth of water over fire boxes, ins.	7.322	6.856	5.629	5.682	5.668	5.533	5.565	5.399
4 " temp. of waste gases, Fahr.	707.910°	696.453°	545.547°	559.760°	566.250°	541.984°	601.047°	575.969°
5 " draft, gauge in chimney, ins.	0.406	0.392	0.438	0.487	0.437	0.429	0.445	0.434
6 " temp. of air during test, Fahr.	85.530°	84.500°	50.812°	42.687°	64.820°	65.719°	69.422°	72.828°
7 " dryness of air " " Fahr.	11.430°	9.330°	5.117°	6.437°	3.047°	11.672°	12.547°	12.125°
8 " barometer " " ins.	29.530	29.570	29.399	29.733	29.569	29.780	29.774	29.762
9 " temp. of feed water, Fahr.	183.870°	186.600°	193.640°	194.514°	191.700°	195.658°	195.184°	194.742°
10 Commercially dry coal burned, lbs.	19 818.00	21 166.00	19 244.00	20 415.00	20 987.00	19 741.00	21 061.00	22 176.00
11 Ash and clinker, weighed back dry, lbs.	1 322.00	2 653.00	1 171.00	1 786.00	1 886.00	1 390.00	1 199.00	2 874.00
12 Percentage of non-combustible, %	6.671	12.534	6.085	8.748	8.986	7.041	5.693	12.960
13 Water supplied to boilers during test, lbs.	173 095.016	172 178.221	174 942.850	176 368.465	175 528.289	173 131.799	172 204.013	175 974.437
14 Water entrained in the steam, corrected, %	0.965	0.965	0.965	0.965	0.965	0.965	0.965	0.965
15 Efficiency of the steam, corrected	0.993	0.993	0.993	0.993	0.993	0.993	0.993	0.993
16 Net water converted into steam, lbs.	171 883.351	170 972.973	173 718.250	175 133.886	174 299.591	171 919.876	170 998.585	174 742.616
17 Steam per lb. of coal from. temp. of feed, lbs.	8.673	8.078	9.027	8.579	8.305	8.709	8.119	7.880
18 Steam per lb. of coal from and at 212° Fahr., lbs.	9.280	8.620	9.566	9.066	8.818	9.211	8.591	8.341
19 Steam per lb. of combustible from temp. of feed, lbs.	9.293	9.235	9.612	9.401	9.125	9.368	8.609	9.053
20 Steam per lb. of combustible from and at 212° Fahr., lbs.	9.943	9.854	10.185	9.935	9.688	9.908	9.110	9.583
21 Steam per sq. ft. of heating surface, per hour, lbs.	5.335	5.340	5.454	5.499	5.472	5.398	5.369	5.486
22 Coal per sq. ft. of grate surface, per hour, lbs.	16.914	18.262	16.613	17.623	18.118	17.041	18.181	19.144
23 Relative efficiency on coal	0.908	0.844	0.936	0.887	0.863	0.902	0.841	0.816
24 Relative efficiency on combustible	0.923	0.915	0.946	0.922	0.899	0.920	0.846	0.890

TABLE No. 9.—West Virginia Coals.

	Kind of coal	Thacker.				Logan Cons. Thacker.	Maritime Thacker.	Campbell's Creek.	
	Grade of coal	Lump.	Mine Run.	Mine Run.	Nut & Slack	Mine Run.	Mine Run.	Mine Run.	Nut & Slack
	Number of test	37	28	50	30	42	46	29	19
	Date of test	Nov. 22, '97.	Oct. 11, '97.	Dec. 30, '97.	Oct. 15, '97.	Dec. 4, '97.	Dec. 13, '97.	Oct. 13, '97.	Sept. 18, '97.
1	Duration of test	15 hrs. 30 m.	16 hrs.	15 hrs. 41 m.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.
2	Av. steam pressure, lbs	101.919	101.766	101.375	101.504	102.281	101.602	101.836	102.000
3	" depth of water over fire boxes, ins	5.663	5.431	5.266	5.443	5.568	5.701	5.515	5.571
4	" temp. of waste gases, Fahr	544.274°	567.281°	456.094°	589.516°	590.547°	568.047°	589.406°	646.187°
5	" draft, gauge in chimney, ins	0.458	0.432	0.421	0.452	0.465	0.470	0.445	0.453
6	" temp. of air during test, Fahr	47.194°	70.156°	43.062°	76.281°	41.773°	45.219°	70.914°	65.109°
7	" dryness of air " " Fahr	2.726°	4.836°	2.539°	9.953°	1.876°	2.367°	9.234°	8.531°
8	" barometer " " ins	29.567	29.187	29.417	29.808	29.391	29.272	29.801	29.518
9	" temp. of feed water, Fahr	193.450°	195.862°	192.699°	197.240°	196.667°	194.412°	195.659°	195.787°
10	Commercially dry coal burned, lbs	18 804.00	20 074.00	19 063.00	20 310.00	19 628.00	19 851.00	20 634.00	22 871.00
11	Ash and clinker, weighed back dry, lbs	1 028.00	1 464.00	1 145.00	2 174.00	1 291.00	851.00	1 570.00	2 078.00
12	Percentage of non-combustible, %	5.467	7.293	6.007	10.704	6.592	4.287	7.609	9.085
13	Water supplied to boilers during test, lbs	169 077.320	179 581.271	176 216.941	175 119.744	171 325.522	174 899.655	175 336.942	172 037.652
14	Water entrained in the steam, corrected %	0.965	0.965	0.965	0.965	0.965	0.965	0.965	0.965
15	Efficiency of the steam, corrected	0.993	0.933	0.993	0.993	0.993	0.993	0.993	0.993
16	Net water converted into steam, lbs	167 893.779	178 324.202	174 983.422	173 893.906	170 126.243	173 675.357	174 104.618	170 833.388
17	Steam per lb. of coal from temp. of feed, lbs	8.929	8.883	9.179	8.562	8.667	8.749	8.438	7.469
18	Steam per lb. of coal from and at 212° Fahr., lbs	9.464	9.592	9.735	9.040	9.158	9.264	8.924	7.899
	(Corrected)			(9.521)					
19	Steam per lb. of combustible from temp. of feed, lbs	9.445	9.582	9.766	9.588	9.279	9.141	9.133	8.216
20	Steam per lb. of combustible from and at 212° Fahr. lbs	10.011	10.132	10.357	10.124	9.804	9.679	9.659	8.689
	(Corrected)			(10.129)					
21	Steam per sq. ft. of heating surface, per hr., lbs.	5.441	5.599	5.605	5.460	5.341	5.453	5.466	5.364
	(Corrected)			(5.482)					
22	Coal per sq. ft. of grate surface, per hour, lbs.	16.756	17.329	16.677	17.533	16.944	17.137	17.812	19.744
23	Relative efficiency on coal	0.926	0.919	0.932	0.885	0.896	0.907	0.874	0.773
24	Relative efficiency on combustible	0.929	0.941	0.940	0.910	0.910	0.899	0.897	0.807

TABLE No. 10.—West Virginia Coals.

	Kind of coal	Monongah.	Eagle.	Cedar Grove.	Mt. Carbon.	Chesapeake.	Coalburg.	Belmont Splint.	
	Grade of coal	Mine Run.	Mine Run.	Mine Run.	Mine Run.	Mine Run.	Mine Run.	Mine Run.	Nut& Slack
	Number of test	36	51	52	53	54	55	38	39
	Date of test	Nov. 20, '97.	Mar. 29, '98.	Mar. 31, '98.	April 2, '98.	April 5, '98.	April 7, '98.	Nov. 24, '97.	Nov. 26, '97.
1	Duration of test	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.
2	Av. steam pressure, lbs	101.625	97.949	99.254	100.308	100.629	100.441	102.004	101.406
3	" depth of water over fire boxes, ins	5.736	5.551	5.478	5.701	5.711	5.672	5.744	5.719
4	" temp. of waste gases, Fahr	548.984°	436.094°	531.094°	549.844°	539.690°	551.484°	570.781°	585.938°
5	" draft, gauge in chimney, ins	0.424	0.390	0.458	0.458	0.458	0.454	0.495	0.462
6	" temp. of air during test, Fahr	58.406°	45.961°	41.406°	47.672	34.549°	46.047°	41.531°	55.047°
7	" dryness of air " " Fahr	7.844°	5.844°	4.945°	6.984°	5.625°	6.985°	4.984°	2.578°
8	" barometer " " ins	29.523	29.875	29.788	29.651	29.709	29.841	29.801	29.441
9	" temp. of feed water, Fahr	193.421°	198.382°	196.432°	194.630°	192.549°	195.304°	194.627°	193.516°
10	Commercially dry coal burned, lbs	20 479.00	19 307.00	18 595.00	19 464.00	19 626.00	19 506.00	21 174.00	22 516.00
11	Ash and clinker weighed back dry, lbs	1 375.00	1 622.00	898.00	1 331.00	1 075.00	1 376.00	1 661.00	2 722.00
12	Percentage of non-combustible, %	6.714	8.401	4.829	6.838	5.477	7.054	7.845	12.089
13	Water supplied to boilers during test, lbs	173 943.320	170 602.293	169 779.607	168 968.583	167 985.900	166 036.184	176 043.283	174 437.084
14	Water entrained in the steam, corrected %	0.965	0.965	0.965	0.965	0.965	0.965	0.965	0.965
15	Efficiency of the steam, corrected	0.993	0.993	0.993	0.993	0.993	0.993	0.993	0.993
16	Net water converted into steam, lbs	172 725.717	169 408.077	168 591.150	167 785.803	166 809.999	164 873.931	174 810.980	173 216.024
17	Steam per lb. of coal from temp. of feed, lbs	8.434	8.774	9.066	8.620	8.499	8.452	8.256	7.693
18	Steam per lb. of coal from and at 212° Fahr., lbs. (Corrected)	8.939	9.248 (9.044)	9.576 (9.365)	9.121 (8.920)	9.014 (8.816)	8.939 (8.742)	8.730	8.152
19	Steam per lb. of combustible from temp. of feed, lbs	9.041	9.579	9.527	9.253	8.992	9.094	8.959	8.751
20	Steam per lb. of combustible from and at 212° Fahr., lbs. (Corrected)	9.582	10.096 (9.874)	10.062 (9.841)	9.791 (9.576)	9.536 (9.326)	9.618 (9.406)	9.485	9.273
21	Steam per sq. foot of heating surface, per hr., lbs. (Corrected)	5.423	5.319 (5.202)	5.293 (5.177)	5.268 (5.152)	5.237 (5.122)	5.176 (5.062)	5.488	5.438
22	Coal per sq. foot of grate surface, per hr., lbs	17.679	16.667	16.052	16.802	16.942	16.839	18.278	19.437
23	Relative efficiency on coal	0.875	0.885	0.917	0.873	0.863	0.856	0.855	0.799
24	Relative efficiency on combustible	0.890	0.917	0.914	0.889	0.866	0.873	0.881	0.861

TABLE No. 11.—Ohio, Kentucky and Tennessee Coals.

Kind of coal	Luhrig-Washed.		Wellston-Shaft.		Jellico—Kentucky.		Jellico-Tenn.
Grade of coal	Egg.	Slack.	Mine Run.	Nut & Slack	Mine Run.	Nut & Slack	Mine Run.
Number of test	41	43	44	45	47	40	48
Date of test	Dec. 1, '97.	Dec. 6, '97.	Dec. 8, '97.	Dec. 10, '97.	Dec. 15, '97.	Nov. 29, '97.	Dec. 17, '97.
1 Duration of test	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.
2 Av. steam pressure, lbs.	102.246	101.539	101.281	100.867	101.454	102.234	101.484
3 " depth of water over fire boxes, ins	5.676	5.588	5.549	5.689	5.521	5.830	5.523
4 " temp. of waste gases, Fahr.	565.860°	612.578	650.937°	652.188°	613.906°	581.250°	633.281°
5 " draft, gauge in chimney, ins.	0.479	0.479	0.484	0.479	0.497	0.506	0.499
6 " temp. of air during test, Fahr.	37.125°	39.352°	52.344°	61.516°	42.914°	34.234°	38.414°
7 " dryness of air " " Fahr.	4.250°	2.493°	3.172°	5.103°	2.875°	3.867°	1.797°
8 " barometer " " ins.	29.606	20.479	29.514	29.482	29.512	29.870	29.667
9 " temp. of feed water, Fahr.	192.606°	196.351°	195.911°	195.176°	194.924°	194.160°	197.505°
10 Commercially dry coal burned, lbs.	23 211.00	24 541.00	22 251.00	23 475.00	20 283.00	23 159.00	21 147.00
11 Ash and clinker weighed back dry, lbs.	1 611.00	1 255.00	665.00	1 104.00	680.00	2 441.00	662.00
12 Percentage of non-combustible, %	6.941	5.114	2.989	4.703	3.352	10.540	3.130
13 Water supplied to boilers during test, lbs.	172 327.338	170 140.303	170 685.354	169 160.625	175 926.554	171 270.734	175 901.457
14 Water entrained in the steam, corrected %	0.965	0.965	0.965	0.965	0.965	0.965	0.965
15 Efficiency of the steam, corrected	0.993	0.993	0.993	0.993	0.993	0.993	0.993
16 Net water converted into steam, lbs.	171 121.047	168 949.321	169 490.556	169 976.501	174 695.068	170 071.839	174 670.147
17 Steam per lb. of coal from temp. of feed, lbs.	7.372	6.884	7.617	7.156	8.613	7.344	8.260
18 Steam per lb. of coal from and at 212°, Fahr., lbs.	9.820	7.275	8.053	7.570	9.115	7.779	8.719
19 Steam per lb. of combustible from temp. of feed, lbs.	7.922	7.255	7.852	7.509	8.912	8.209	8.527
20 Steam per lb. of combustible from and at 212° Fahr., lbs.	8.404	7.667	8.301	7.943	9.431	8.695	9.001
21 Steam per sq. foot of heating surface, per hour, lbs.	5.373	5.304	5.321	5.274	5.485	5.340	5.484
22 Coal per sq. foot of grate surface, per hour, lbs.	20.037	21.186	19.209	20.265	17.509	19.991	18.255
23 Relative efficiency on coal	0.765	0.712	0.788	0.741	0.892	0.761	0.853
24 Relative efficiency on combustible	0.789	0.712	0.771	0.737	0.876	0.807	0.836

The relative efficiency of the several coals, on coal (23), and on combustible (24) are based upon the highest evaporation obtained during the tests per pound of coal or combustible from and at 212° Fahr. as standards, and the factors for multiplying the prices at which several coals may be offered are, of course, the reciprocals of these efficiencies. If thus, the coals mentioned in Table No. 12 were offered at the prices per ton annexed, then column 3 contains the deduced prices, obtained in the manner described.

TABLE No. 12.

Kind of coal.	Dealers' price per ton.	Deduced price for comparison of bids.
Pittsburg, Brown, Second Pool, Mine Run	$1.50	$1.82
Pittsburg, Catsburg, Third Pool, Mine Run	1.75	2.10
Pittsburg, Woods, Fourth Pool, Mine Run	1.60	1.83
Pocahontas, Mine Run	1.75	1.90
New River, Mine Run	1.64	1.73
Wellston Shaft, Mine Run	1.90	2.41

Upon these prices, New River coal would be offered at the lowest cost to the steam user.

The reciprocals of the efficiencies based on coal also show how much more than the standard will be required of any of the coals tested to do a given amount of work.

The economic and capacity results of the Pennsylvania coals are shown in Table No. 13.

The relative values of the grades of Second Pool coal noted in the table, from the dealers' point of view, are about as follows:

Lump	100
Barge Run	90–95
Mine Run	80–90
Nut and Slack	70–80

The grades of coal show differences in value to the buyer or steam user as follows:

Lump	100.00
Barge Run	100.67
Mine Run	93.56
Nut and Slack	93.72

TABLE No. 13.—Economic and Capacity Results with Pennsylvania Coals.

Test number.	Kind of coal.	Steam from and at 212° Fahr.		Steam per square foot of heating surface per hour.	Coal per square foot of grate surface per hour.
		Per pound of coal.	Per pound of combustible.		
		Pounds.	Pounds.	Pounds.	Pounds.
13..	Pittsburg, Brown, Second Pool, Lump.....	9.004	9.537	5.397	17.408
15..	" " " Barge Run..	9.064	9.642	5.398	17.297
14..	" " " Mine Run...	8.424	9.251	5.358	18.484
16..	" " " Nut and Slack.....	8.439	9.241	5.413	18.666
5..	Pittsburg, Catsburg, Third Pool, Lump.....	9.227	9.868	5.317	16.783
6..	" " " Barge Run	8.790	9.459	5.419	17.960
7..	" " " Mine Run.	8.521	9.364	5.391	18.426
8..	" " " Nut and Slack....	8.227	9.143	5.334	18.839
17..	Pittsburg, Cincinnati, Third Pool, Mine Run	8.673	9.237	5.362	17.990
18..	" " " Nut and Slack....	8.672	9.344	5.391	18.061
9..	Pittsburg, Woods, Fourth Pool, Lump.....	8.735	9.648	5.375	17.875
10..	" " " Barge Run.	8.519	9.426	5.398	18.432
11..	" " " Mine Run..	8.933	9.903	5.407	17.593
12..	" " " Nut and Slack....	8.452	9.426	5.377	18.468

With this coal it appeared that there was no superiority of Lump over Barge Run coal, while Mine Run and Nut and Slack were less than 6½% inferior in economy to Lump coal; and Nut and Slack coal for steam purposes, at the prices usually asked, was superior to Lump coal by 26.20%; or while one dollar's worth of Second Pool, Lump coal furnished 10 290 lbs. of steam, one dollar's worth of Nut and Slack coal furnished 12 983 lbs. of steam, and one dollar's worth of Mine Run coal furnished 11 232 lbs. of steam.

The Third Pool coals, for the different grades, have the following relative mean values:

Lump	100.00
Barge Run	92.35
Mine Run	93.17
Nut and Slack	92.12

The Fourth Pool coals have the following values for the different grades:

Lump	100.00
Barge Run	97.53
Mine Run	102.27
Nut and Slack	96.76

TABLE No. 14.—Economic and Capacity Results with West Virginia Coals.

Test No.	Kind of coal.	Steam from and at 212° Fahr.		Steam per square foot of heating surface per hour.	Coal per square foot of grate surface per hour.
		Per pound of coal.	Per pound of combustible.		
		Pounds.	Pounds.	Pounds.	Pounds.
35....	New River, Lump	9.186	9.879	5.400	17.149
49....	" Lump	9.707	10.143	5.333	15.590
1....	" Mine Run	9.834	10.771	5.319	15.794
31....	" Mine Run	9.572	10.005	5.477	16.811
27....	Loup Creek, Lump*	10.215	10.633	5.567	15.837
20....	" Mine Run†	8.759	9.130	5.434	18.048
21....	" Slack*	8.776	9.216	5.216	17.272
26....	Pocahontas, Lump	9.958	10.603	5.353	15.645
22....	" Mine Run	9.423	10.133	5.262	16.247
2....	Kanawha, Steam Lump	9.280	9.943	5.335	16.914
3....	" Gas Lump	8.620	9.854	5.340	18.262
32....	Acme, Mine Run	9.566	10.185	5.454	16.613
33....	" Nut and Slack	9.066	9.935	5.499	17.623
34....	Keystone, Mine Run	8.818	9.688	5.472	18.118
23....	Winifrede, Lump	9.211	9.908	5.398	17.041
24....	" Mine Run	8.591	9.110	5.369	18.181
25....	" Nut and Slack	8.341	9.583	5.486	19.144
37....	Thacker, Lump	9.464	10.011	5.441	16.756
28....	" Mine Run	9.392	10.132	5.599	17.329
50....	" Mine Run	9.521	10.129	5.482	16.677
30....	" Nut and Slack	9.040	10.124	5.460	17.533
42....	Logan Consolidated, Thacker, Mine Run	9.158	9.804	5.341	16.944
46....	Maritime, Thacker, Mine Run	9.264	9.679	5.453	17.137
29....	Campbell's Creek, Mine Run	8.924	9.659	5.466	17.812
19....	" " Nut and Slack	7.899	8.689	5.364	19.744
36....	Monongah, Mine Run	8.939	9.582	5.423	17.679
51....	Eagle, Mine Run	9.044	9.874	5.202	16.667
52....	Cedar Grove, Mine Run	9.365	9.841	5.177	16.052
53....	Mount Carbon, Mine Run	8.920	9.576	5.152	16.802
54....	Chesapeake, Mine Run	8.816	9.326	5.122	16.942
55....	Coalburg, Mine Run	8.742	9.406	5.062	16.839
38....	Belmont Splint, Mine Run	8.730	9.485	5.488	18.278
39....	" " Nut and Slack	8.152	9.273	5.438	19.437

* From the MacDonald Mine.
† From the Dunn Loup Mine.

An examination of these figures shows two things:

First.—That the opinion hitherto generally accepted as to Second Pool, Monongahela River coal being markedly superior to the coals obtained from the newer Pittsburg mines farther up the river, is not sustained by the results of these tests.

Second.—That the prices usually asked for the higher grades of these coals are not warranted for steam purposes; that Mine Run and Nut and Slack coals are by far the most economical coals, and that they can be sold upon narrower discounts from Lump coal prices and still be the most profitable to buyers of steam coals.

Referring briefly to the economic results per pound of combustible, it will be noticed that there is greater uniformity among all these coals in the steam per pound of combustible, without regard to the mines or grades, than in the steam per pound of coal, and this naturally is due to the fact that the lower grades contain just as good coal as Lump, but carry more waste matter from the slate and stone loosened in mining from the roof and floor of the galleries, which in Lump coal is presumed to be removed when it is run over the screen at the mouth of the mine, and by the use of finer screens often goes into the cargoes of low-grade coals, and is purchased as so much coal by the buyer. Assuming no more non-combustible matter in the Mine Run than in the Lump coal, such coal for steam purposes should always be equal to Lump, provided the grates upon which it is burned are suitable to retain the smaller and finer particles until coking occurs, or in the non-coking coals, until combustion is completed.

A review of the Pittsburg coals indicates that they should be rated as of equal quality, whether obtained from the older mines in the Second Pool or from the newer mines in the Fourth Pool.

The economic and capacity results of the West Virginia coals are shown in Table No. 14.

The economic and capacity results of the Ohio, Kentucky and Tennessee coals are shown in Table No. 15.

TABLE No. 15.—Economic and Capacity Results with Ohio, Kentucky and Tennessee Coals.

Test No.	Kind of coal.	Steam from and at 212° Fahr.		Steam per square foot of heating surface per hour.	Coal per square foot of grate surface per hour.
		Per pound of coal.	Per pound of combustible.		
		Pounds.	Pounds.	Pounds.	Pounds.
	Ohio Coals.				
41	Luhrig, Washed Egg	7.820	8.404	5.373	20.037
43	Luhrig, Washed Slack	7.275	7.667	5.304	21.186
44	Wellston Shaft, Mine Run	8.053	8.301	5.321	19.209
45	Wellston Shaft, Nut and Slack	7.570	7.943	5.274	20.265
	Kentucky Coals.				
47	Jellico, Mine Run	9 115	9.431	5.485	17.509
40	Jellico, Nut and Slack	7.779	8.695	5.340	19.991
	Tennessee Coal.				
48	Jellico, Mine Run	8.719	9.001	5.484	18.255

A hasty glance at the table of economic results obtained with the West Virginia coals will show that nearly all the samples should be ranked higher than the Pittsburg coals. Thus the average weight of steam per pound of coal for all the Pittsburg coals was 8.69 lbs., an economy which is reached or excelled by all except five samples of West Virginia coals. Heretofore, in the western river markets, Pittsburg coal from the Second Pool has been the accepted standard, and for years prices and guarantees have been based on this coal.

Considering Pittsburg coal as the standard for steam purposes in the western river markets, all but five of the West Virginia coals exceed the average of the Pittsburg coals in steam per pound of coal and per pound of combustible. Belmont Splint, Winifrede, Campbell's Creek, Kanawha Gas, and the Loup Creek low-grade coals are the only kinds or grades that fall below the Pittsburg coals. Of the West Virginia coals, several, like New River, Pocahontas, Loup Creek (from the MacDonald Mine), Acme, Thacker, Cedar Grove and Kanawha Steam, are peculiarly of good quality.

Grouping the West Virginia coals in two classes, of which the first contains coals distinctly superior to Pittsburg, and the second all other coals tested, the average performance for the coals of each of these classes is shown in Table No. 16.

TABLE No. 16.

Samples of coals.	Steam from and at 212° Fahr.	
	Per pound of coal. Pounds.	Per pound of combustible. Pounds.
Pittsburg coals (average)	8.69	9.46
First-class, West Virginia coals: New River, Loup Creek (MacDonald Mine), Pocahontas, Kanawha Steam, Acme, Thacker, and Cedar Grove coals (average)	9.51	10.16
Second-class, West Virginia coals: Loup Creek (Dunn Loup Mine), Kanawha Gas, Keystone, Winifrede, Logan Consolidated Thacker, Maritime Thacker, Campbell's Creek, Monongah, Eagle, Mt. Carbon, Chesapeake, Coalburg and Belmont Splint coals (average)	8.76	9.49

By referring to the proximate analyses and calorific tests of the seven coals grouped in the first class, it will be noticed that the composition of the combustible matter varies widely, while the heating

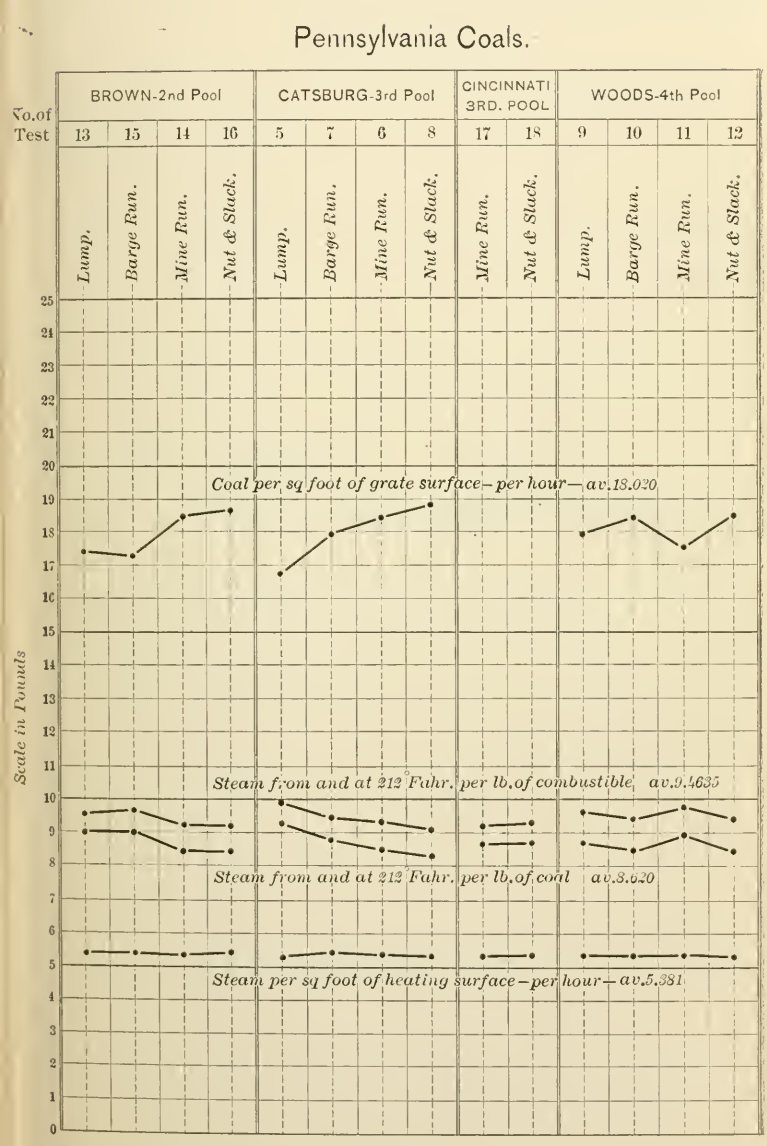

Pennsylvania Coals.
BROWN-2nd Pool
CATSBURG-3rd Pool
CINCINNATI 3RD. POOL
WOODS-4th Pool
No. of Test
13 15 14 16 5 7 6 8 17 18 9 10 11 12
Lump. Barge Run. Mine Run. Nut & Slack. Lump. Barge Run. Mine Run. Nut & Slack. Mine Run. Nut & Slack. Lump. Barge Run. Mine Run. Nut & Slack.
Coal per sq foot of grate surface—per hour—av.18.020
Steam from and at 212° Fahr. per lb. of combustible av.9.4635
Steam from and at 212° Fahr. per lb. of coal av.8.620
Steam per sq foot of heating surface—per hour—av.5.381
Scale in Pounds
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25

FIG. 3.

powers of combustible and practical performance are sufficiently near together to suggest coals of like character.

TABLE No. 17.

Sample of coal.	B. T. U. per pound of combustible.	Steam per pound of combustible (averages).
New River	15 757	10.20
Loup Creek (MacDonald Mine)	15 336	10.63
Pocahontas	15 456	10.37
Kanawha Steam	15 653	9.94
Acme	15 589	10.06
Thacker	15 554	10.10
Cedar Grove	15 422	9.84

From the proximate analyses of these coals, the averages of fixed carbon and volatile matter are as follows:

Sample of coal.	Fixed carbon.	Volatile matter.
New River, Loup Creek and Pocahontas	78.77	18.85
Kanawha Steam, Acme and Thacker	64.72	31.37
Cedar Grove	60.26	35.35

Referring to the Ohio, Kentucky and Tennessee coals tested, it is apparent that for steam purposes these coals are not equal to the West Virginia and Pittsburg coals. (An exception should be made of the Jellico, Ky., coal, which compares favorably with the second-class, West Virginia coals.)

The diagrams (Figs. 3 to 6, inclusive) show the relation of the different coals of each group and grade. The vertical scale represents pounds, and the horizontal scale the different coals. The solid circles connected by lines at the top of the diagrams indicate (to scale) the average weights of coal samples burned per square foot of grate surface per hour. The second series of circles connected by lines indicates the steam per pound of combustible; and the third series of circles connected by lines indicates the steam per pound of coal, both from and at 212° Fahr. The lower solid circles and lines indicate the steam in pounds per square foot of heating surface per hour. The reasonable uniformity of the work done during each test by all coals will be noticed upon reference to the several diagrams.

It is quite possible that the boilers used in these tests were not the best for all the coals tested, and that some kinds and grades of coal

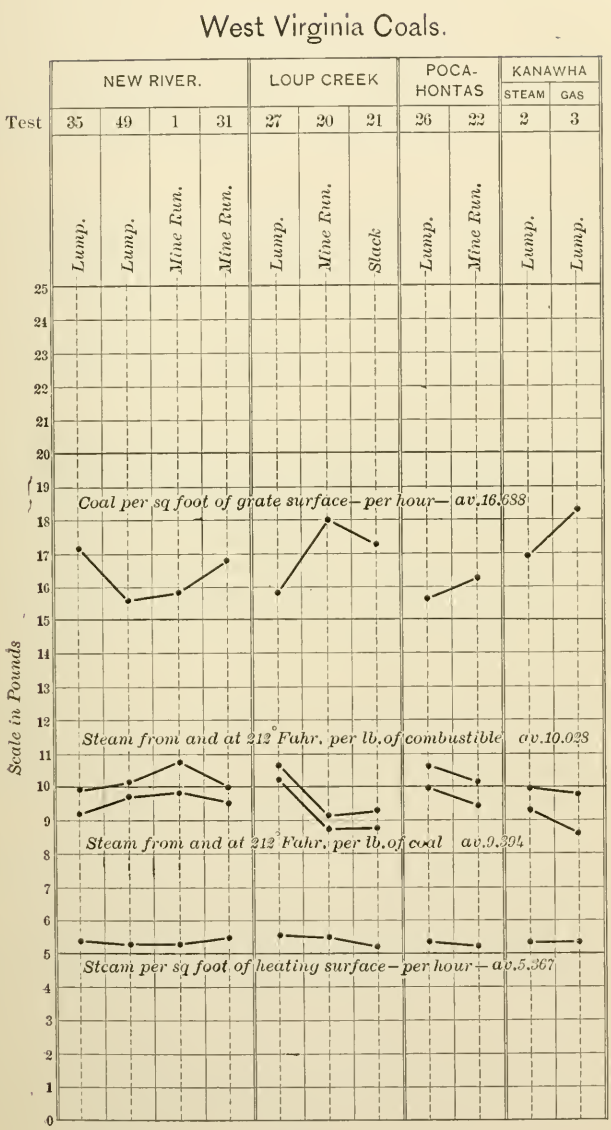

West Virginia Coals.
NEW RIVER.
LOUP CREEK
POCA-HONTAS
KANAWHA
STEAM
GAS
No.of Test
35
49
1
31
27
30
21
26
22
2
3
Lump.
Lump.
Mine Run.
Mine Run.
Lump.
Mine Run.
Slack
Lump.
Mine Run.
Lump.
Lump.
Coal per sq foot of grate surface—per hour—av.16.638
Steam from and at 212° Fahr. per lb. of combustible av.10.028
Steam from and at 212° Fahr. per lb. of coal av.9.394
Steam per sq foot of heating surface—per hour—av.5.367
Scale in Pounds
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25

FIG. 4.

suffered accordingly; but that is a condition which perhaps will always be found true, for no form of boiler and furnace can be equally well adapted to every variety of bituminous coal, nor could changes be made in the boiler and furnace, and in the mode of firing, for different kinds of coal, without raising a suspicion in the investigator's mind that by so doing he would be favoring one coal at the expense of another.

The Galloway boilers have elsewhere been reported upon very favorably when worked with bituminous coal, results being attributed to them with what the author finds to be inferior coals, which considerably exceed the performance of any coal tested during this investigation, but doubtless such results were obtained under conditions which cannot be had in the daily operation of steam boilers with ordinary firemen.

These boilers were originally set with 960 sq. ft. of heating surface and 199 sq. ft. of super-heating surface, a total combined heating and super-heating surface of 1 159 sq. ft. for each boiler. The grate surface, of course, was the same as during the author's trials, but the chimney was originally 150 ft. high and of the same section as during the later tests. The report of the contract trial upon which the boilers were accepted by the city contains the following data:

Date of trial	May, 1889.
Duration of trial	10 hrs.
Steam pressure, by gauge	92.80 lbs.
Temperature of feed water	193.70° Fahr.
Temperature of waste gases in flue leading to chimney*	700° + Fahr.
Dry coal consumed	6 911.00 lbs.
Super-heat, expressed as percentage gain of heat units above saturated steam (no data given)	11.03%
Total steam from and at 212° Fahr	76 377.00 lbs.
Steam per pound of dry coal from and at 212° Fahr	11.05 lbs.

The coal for the contract trial is stated as Pittsburg Nut, containing 10.3% of ash, etc., and over 3¼% of moisture. These percentages are much larger than found by the author for any kind or grade of Pittsburg coal. The coal was burned at the rate of 19.05 lbs. per square

* Obtained by melting zinc.

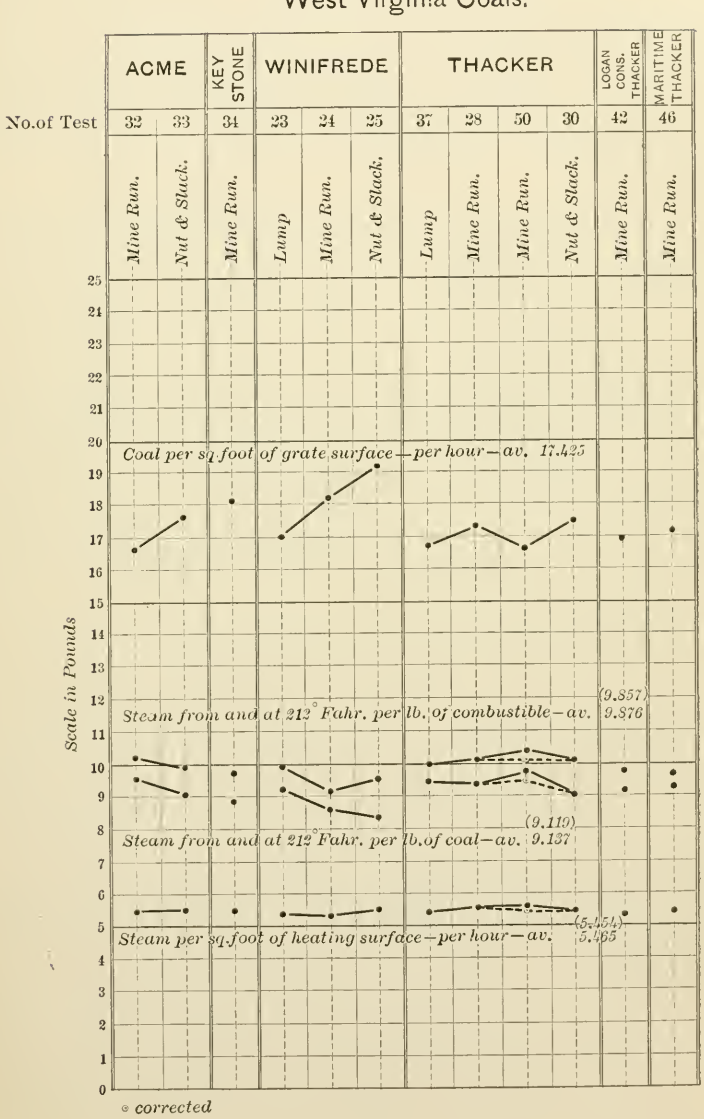

West Virginia Coals.
ACME
KEY STONE
WINIFREDE
THACKER
LOGAN CONS. THACKER
MARITIME THACKER
No.of Test
32
33
34
23
24
25
37
38
50
30
42
46
Mine Run.
Nut & Slack.
Mine Run.
Lump
Mine Run.
Nut & Slack.
Lump
Mine Run.
Mine Run.
Nut & Slack.
Mine Run.
Mine Run.
Scale in Pounds
Coal per sq. foot of grate surface—per hour—av. 17.425
Steam from and at 212° Fahr. per lb. of combustible—av. 9.876
(9.857)
Steam from and at 212° Fahr. per lb. of coal—av. 9.137
(9.119)
Steam per sq. foot of heating surface—per hour—av. 5.465
(5.454)
corrected

FIG. 5.

foot of grate surface per hour, and steam was made at the rate of 8.03 lbs. from and at 212° Fahr. per square foot of water-heating surface per hour. The principal differences between the conditions for the contract trial, and those for the coal tests by the author, were in the setting of the boilers and in the height of the chimney, with the complete loss of super-heating surface for the later trials, which apparently alone made the efficiency of the steam 11.73% better for this earlier trial.

The author has no record of the methods pursued, and no means of checking the results of the contract trial, but it would be interesting to know how the very high percentage of super-heat was obtained with a boiler setting which within his experience has seldom shown above a few degrees of super-heat, and generally is expected by the builders only to secure dry steam, and further how the largely increased capacity was obtained. During the author's tests the boilers were supposed to be doing the best possible work, with an economic rate of coal consumption per square foot of grate surface per hour. If there was error in reporting the contract trial, it would seem to be either in the weight of water credited to the boiler, or in the quality of steam made, or both. The rate of coal consumption compares fairly well with the rates used during the author's coal tests.

In all instances the coals were used with care, and the furnace conditions were such as, in the author's opinion, would give the coal under trial a fair rating. One circumstance, however, seemed to militate against all coals, *i. e.*, the very high temperature of the chimney gases, which suggested at times that the coals were being burned at too high a rate per unit of grate surface, or that the ratio of grate and heating surface was not well proportioned. The higher temperatures of hot gas were obtained during the earlier tests, and this was then thought to have been due to the fireman's lack of previous experience with these boilers, but singularly enough some of the high temperatures in the chimney were accompanied by better economies than when the same coals were tried with lower temperatures of waste gases. By referring to the abstract from the reported results of the contract trial, it seems that the waste gases entered the chimney then at a very high temperature, notwithstanding the unusual economic results obtained.

During the first 22 tests there was no regulation of the flow of air to

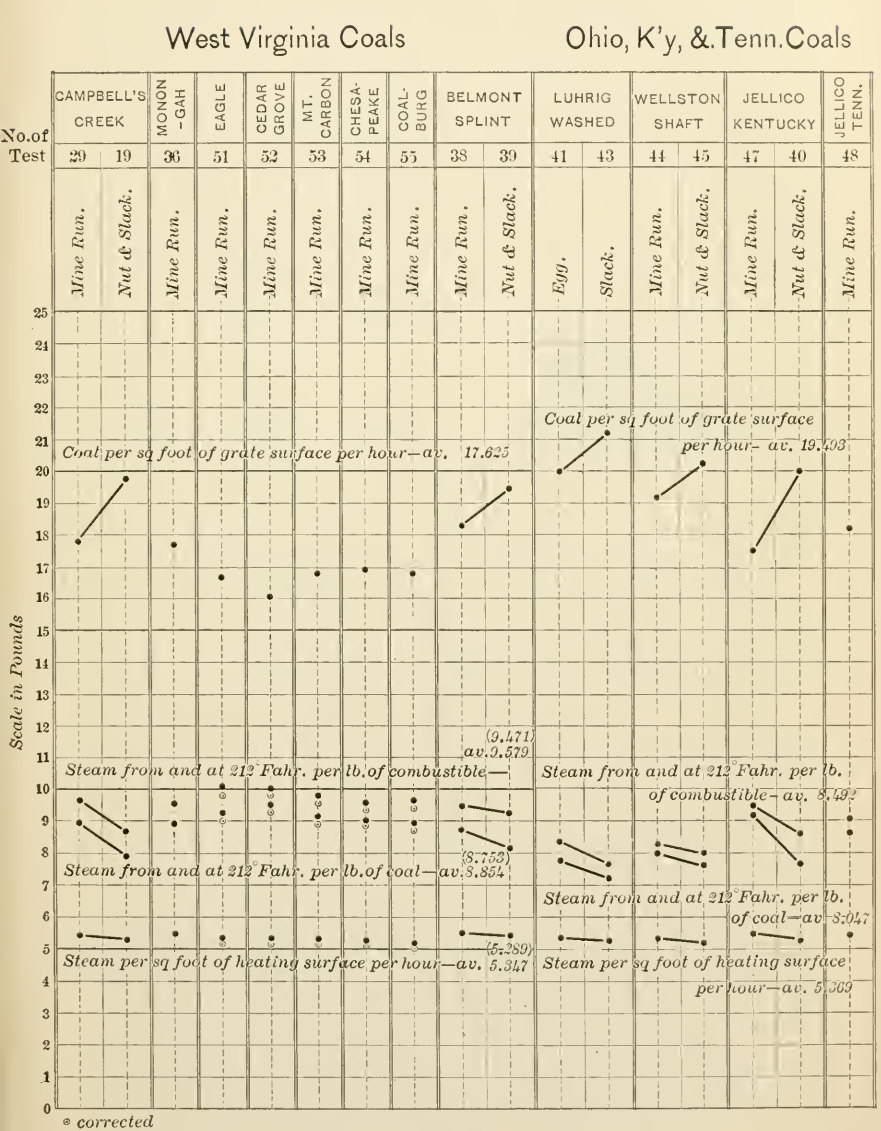

West Virginia Coals
Ohio, K'y, &.Tenn.Coals
CAMPBELL'S CREEK
MONON-GAH
EAGLE
CEDAR GROVE
MT. CARBON
CHESA-PEAKE
COAL-BURG
BELMONT SPLINT
LUHRIG WASHED
WELLSTON SHAFT
JELLICO KENTUCKY
JELLICO TENN.
No.of Test
29 19 36 51 52 53 54 55 38 39 41 43 44 45 47 40 48
Mine Run. Nut & Slack. Mine Run. Mine Run. Mine Run. Mine Run. Mine Run. Mine Run. Mine Run. Nut & Slack. Egg. Slack. Mine Run. Nut & Slack. Mine Run. Nut & Slack. Mine Run.
Coal per sq foot of grate surface per hour—av. 17.625
Coal per sq foot of grate surface per hour—av. 19.493
Steam from and at 212° Fahr. per lb. of combustible—
(9.471) av. 9.579
Steam from and at 212° Fahr. per lb. of combustible—av. 8.493
(8.753)
Steam from and at 212° Fahr. per lb. of coal—av. 8.854
Steam from and at 212° Fahr. per lb. of coal—av. 8.047
(5.289)
Steam per sq foot of heating surface per hour—av. 5.347
Steam per sq foot of heating surface per hour—av. 5.369
Scale in Pounds
25 24 23 22 21 20 19 18 17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1 0
⊙ corrected

FIG. 6.

the ash-pits, and comparison of coals of like kinds tried before and after the quantity of air to the ash-pits was placed under control, indicates in one example a gain, and in the other a slight loss by the change.

TABLE No. 18.—Comparison of Coals tried before and after Regulation of Air to Ashpits.

Test Number.	Kind of coal.	Steam from and at 212° Fahr.		Temperature of waste gases.
		Per pound of coal. Pounds.	Per pound of combustible. Pounds.	Fahr.
1	New River Mine Run.....	9.834	10.771	631.3
20	Loup Creek, Mine Run.....	8.759	9.130	718.8
31	New River, Mine Run	9.572	10.005	521.7
35	New River, Lump.....	9.186	9.879	622.6
49	New River, Lump.....	9.707	10.143	606.5
27	Loup Creek, Lump.....	10.215	10.633	637.6

Calorimetric Determinations of Heating Power of Coals.

Before preceding to this branch of the investigation, careful study was made of several modern forms of coal calorimeter, and, after weighing the advantages and disadvantages of each, the author decided to use the instrument devised by Professor R. C. Carpenter, of Cornell University, the accuracy of which, after observing due precautions in its use, depends upon the assumption that equal increments of heat produce corresponding dilatations of the water in the instrument, and that the heat value for pure carbon has been determined. The instrument was accompanied by a diagram, which gave the manufacturer's calibration and value for each inch of rise on the scale.

By burning in the instrument successive weights of pure carbon, obtained by coking recrystallized sugar, the value in heat units of 1 in. on the scale of the water column was obtained, and by multiplying this by the rise in inches and dividing by the weight of fuel burned in fraction of a pound, the heat units per pound of coal or coke sample were obtained. The construction and use of this form of coal calorimeter is fully described in a paper by Professor Carpenter.*

* *Transactions*, A. S. M. E., Vol. xvi, 1895.

The value of pure carbon burning without a residue, as determined by Berthelot, is 14 652 B. T. U. per pound,* while two samples of pure carbon, burning without a residue in the crucible, gave during the author's tests a mean of 14 646 when measured on the heat scale supplied with the Carpenter instrument.

The preliminary tests with coke from sugar were carefully repeated with trifling variations in the results, and, accepting the value of pure carbon as determined by Berthelot, then 1 in. rise of the water column equals 7.15 B. T. U.; *i. e.*, this amount of heat added to the water in the instrument caused the column in the glass tube to rise 1 in. Generally, 1.5 grams of coal were burned in the calorimeter, but as a check on the work 1 gram and 2 grams were sometimes used.

The sample of coal for the calorimeter was weighed on an analytical balance, turning to $\frac{1}{10}$ mgr., in thin asbestos crucibles about $\frac{3}{4}$ in. in diameter and $\frac{3}{4}$ in. deep. These crucibles were subjected to prolonged heat over a Bunsen burner to remove any combustible matter, and cooled to room temperature in a desiccator before each determination.

Sample determinations with this calorimeter are given below:

Pure Carbon from Sugar.—1.5 grams. Scale reading 6.74 ins.

$$\text{B. T. U.} = \frac{6.74 \times 7.15}{0.00331} = 14\,560$$

Harvey Foundry Coke.—1.411 grams. Scale reading 6.15 ins.

$$\text{B. T. U.} = \frac{6.15 \times 7.15}{0.00311} = 13\,871$$

New River, Mine Run, Coal.—1.5 grams. Scale reading 7.02 ins.

$$\text{B. T. U.} = \frac{7.02 \times 7.15}{0.00331} = 15\,163$$

The Carpenter instrument, which is an improved form of the earlier Thompson Gas Calorimeter, is really a very large thermometer; the fuel being burned in the bulb, and all heat being absorbed by the fluid or lost by radiation. Combustion takes place in the presence of pure oxygen under a pressure of 9 ins. of water. The heat absorbed is accounted for by the expansion of the fluid in the vertical glass tube, while that lost by radiation is measured by the contraction of the column of fluid during an interval of time equal to the time required for the combustion of the coal sample. The expansion and contraction are measured in inches on a vertical scale placed behind the glass

*Stillman's "Engineering Chemistry," 1897, p. 120. Poole's "Calorific Power of Fuels," 1898, p. 13.

tube. The bulb of the instrument contained slightly less than 5 lbs. of water at 60° Fahr.

The precautions to be observed in the use of this calorimeter are few and easily complied with:

First.—Distilled water, absolutely free from air, should be used. This the author obtained by first gently boiling the water in a separate vessel placed over a Bunsen gas-burner, and then introducing it at high temperature into the calorimeter bulb. While cooling to nearly room temperature the bulb was so held as to cause any air which may have been trapped while filling, to seek the stuffing-box as an outlet. To prevent reabsorption of air from the atmosphere while working with the calorimeter, a column of coal oil $1\frac{1}{2}$ ins. high was run into the glass tube on top of the water.

Second.—At the beginning of each experiment, the temperature of the calorimeter and water must be sensibly higher than room temperature to avoid transfer of heat from the atmosphere to the water, and the instrument must be rigidly protected from all drafts in the room while the experiment is being conducted.

Third.—The exit orifice for the waste gases must be kept clear of all residual matters which may come over and be deposited from them. With perfect combustion there should be no residuals in the waste gases; but it is essential to good work with this instrument that the gas orifice be frequently opened with a fine brass wire, for with continued use in some manner this very small opening (about the size of a fine cambric needle) becomes clogged. (It is possible that the sulphuric acid in the waste gases may have a corrosive effect on the metal surrounding the exit orifice, with formation of sulphate of copper.)

Fourth.—The time required to burn the sample of coal should be noted to seconds, and the same length of time allowed for loss of heat by radiation after combustion.

Many preliminary tests with this instrument on foundry coke and pure carbon indicated an accuracy of measurement which in the comparative determination of the heat values of these coals should be all sufficient.

In Table No. 19 are given the averages of several determinations of the calorific values in B. T. U. of the different coals subjected to physical tests, together with a comparison of the ash by calorimeter, and by chemical analysis:

TABLE No. 19.—Heating Power of Coals in British Thermal Units.

Test numbers.	Kind of coal.	Heat units per pound of coal.	Ash by calorimeter.	Ash by analysis.
	Pennsylvania (Pittsburg) Coals.			
13-14-15-16.	Pittsburg, Brown, Second Pool	13 383	5.77	3.75
5-6-7-8.	" Catisburg, Third Pool	13 446	5.46	5.50
17-18.	" Cincinnati, Third Pool	13 247	4.64	4.05
9-10-11-12.	" Woods, Fourth Pool	13 316	5.66	5.60
	West Virginia Coals.			
5-31-35-49.	New River (old sample)	14 742	1.40	1.45
1-31-35-49.	" (new sample)	15 163	2.87	2.40
10-21-27.	Loup Creek (old sample)	14 137	4.17	2.15
20-21-27.	" (new sample)	14 975	1.20	0.95
22-26.	Pocahontas (old sample	14 048	4.23	4.00
22-26.	" (new sample)	15 133	1.57	1.25
2.	Kanawha, Steam	15 163	2.33	2.10
2.	" Gas	15 088	1.33	3.75
32-33.	Acme	15 184	1.60	2.10
34.	Keystone	13 824	9.73	8.30
33-24-25.	Winifrede	14 111	2.91	2.85
28-30-37-50.	Thacker	14 526	5.88	5.00
22.	Logan Consolidated Thacker	14 245	4.13	3.85
46.	Maritime Thacker	14 126	5.20	4.60
49-29.	Campbell's Creek	14 189	3.17	2.90
16.	Monongah	14 245	3.04	3.30
31.	Eagle	14 040	3.00	2.60
52.	Cedar Grove	14 668	3.97	3.47
53.	Mt Carbon	13 964	4.67	4.33
54.	Chesapeake	13 370	3.93	3.27
55.	Coalburg	13 781	5.26	4.93
38-39.	Belmont Splint	12 949	1.08	1.20
	Ohio Coals.			
41-43.	Luhrig, Washed	12 960	6.86	6.90
44-45.	Wellston Shaft	12 506	7.80	7.30
	Kentucky Coals.			
40-47.	Jellico	14 472	1.06	0.85
	Tennessee Coal.			
48.	Jellico	13 846	1.37	1.10

The ash, both by calorimeter and analysis, is a mean of several determinations, and, when, by the two methods of examination, the percentages of ash fail to agree, it must be charged to lack of uniformity in the samples. This variation in percentage of ash in coals from the same source is well known,* and indicates, as might be supposed, that Nature in forming the coal did not follow exact synthesis.

Some writers have recently stated the thermal values of certain coals on the net combustible, leaving out of consideration the ash and moisture, which are constant constituents of all coals as purchased for use. The propriety of this, excepting for comparison of effects of different combinations of carbon, hydrogen and oxygen in the coal,

* " The Calorific Value of Certain Coals," etc., by Lord and Haas, *Loc. cit.*, pp. 8 and 9.

may be questioned. Thus, when the constituents of the combustible are alike, or nearly alike, in two or more coals, and the B. T. U. are substantially different, such a comparison furnishes very interesting information; but when the thermal values agree very well for different coals of like composition, it is a mistake to neglect properties which are certain to be found in all coals of every nature. Neither does it furnish information of value to the consumer of coal, who is compelled to make his purchases and estimate economy with the ash and moisture included.

In connection with the heat values of the coals tested by the author, Table No. 20, comparing some of these with results obtained by other investigators on the same coals, may be of interest. In this table the data attributed to Professor Carpenter were obtained with an instrument of the same kind as that used by the author.* The data credited to Mr. Barrus were obtained from an improved form and mode of using the Thompson calorimeter.† The data credited to Professors Lord and Haas were obtained with a Mahler bomb calorimeter,‡ while those credited to Messrs. Hale and Williams were obtained with a bomb calorimeter of the Berthelot type.§

With reference to the samples of coal tested for calorific power by the author, it is proper to state that all the samples were in the office for from two weeks to eight months previous to making the calorimetric determinations, and although excluded from dust, the packages were open to free access of air, and some of the volatile matter may have been lost under the influence of room temperature. To ascertain the probable effect of keeping the coals under this condition, tests of old and new samples of New River, Loup Creek and Pocahontas coals were made, with the result in each instance, of a higher calorific power in the new when compared with the old samples. Upon the contrary, the recent experiments of Messrs. Hale and Williams, Boston, indicate that two of the coals mentioned (New River and Pocahontas) were not impaired in heating power by exposure or weathering. With this difference, however, that exposure in the author's tests was limited to the air circulating in a room, while in Messrs. Hale and Williams' experiments the samples of coal were really exposed to the weather for eleven months.

* *Transactions*, A. S. M. E., Vol. xvi, 1895.
† *Transactions*, A. S. M. E., Vol. xiv, 1893.
‡ *Transactions*, Am. Inst. Mining Engrs., 1897.
§ *Transactions*, A. S. M. E., Vol. xx, 1899.

More thau 20 years ago the author made some experiments under steam boilers with weathered and freshly mined Pittsburg coal, with the result that the freshly mined coal gave the higher amount of steam per pound of coal. The data of these early trials not having been permanently recorded, the percentage gain of the freshly mined over-weathered coal cannot now be stated, but the difference was quite material.

TABLE No. 20.—Comparison of Heat Determinations.

Kind of coal.	Heat units per pound of coal.	Mode of Determination.	Authority.
New River	14 742 (old)	Calorimeter.	Author.
	15 163 (new)		
"	13 858	"	G. H. Barrus.
	14 455		
"	14 713	Analysis.	Hale and Williams.
	15 222		
Pocahontas	14 048 (old)	Calorimeter.	Author.
	15 133 (new)		
"	15 094	"	R. C. Carpenter.
"	14 512	"	Lord and Haas.
	14 906		
"	13 608	"	G. H. Barrus.
	14 603		
"	14 975	Analysis.	Hale and Williams.
Thacker	14 526	Calorimeter.	Author.
"	13 955	"	Lord and Haas.
	14 020		
Pittsburg	13 247	"	Author.
	13 316		
	13 383		
	13 446		
"	13 237	"	Lord and Haas.
	13 977		
"	12 941	"	G. H. Barrus.
"	13 414	Analysis.	Hale and Williams.
	13 657		
	13 963		
	14 006		
Luhrig (Ohio)	12 960	Calorimeter.	Author.
Wellston (Ohio)	12 506	"	"
Hocking Valley (Ohio)	11 885	"	Lord and Haas.
	12 388		

Professor Johnson attempted to deduce the calorific values of the coals which came under his observation from the formulas of Despretz and Dulong* (the then prevailing expressions for the total heating power of coal), and also to relate the total heat value to that of pure carbon by the method of Berthier, which consists in reducing the lead from litharge by combustion of a known weight of coal in a crucible in the presence of an excess of the oxide of lead. The amount

* Report to the Navy Department of the United States, on American Coals, by Professor Walter R. Johnson, 1843, p. 500.

TABLE No. 21.—Calorimeter Experiments with Harvey Coke, Gas Coke, and Pure Carbon from Recrystallized Sugar.

Sample.	Weight in grams.	B. T. U. Per pound of combustible.
Harvey Coke	1.885	13 641
"	1.411	13 860
"	1.418	14 026
"	1.413	13 987
"	0.943	13 600
"	1.885	14 327
"	0.944	13 461
Gas Coke	1.500	13 000
Pure Carbon	1.500	14 560
"	1.590	14 732

of metallic lead found, after combustion, as a button in the crucible, being weighed and related to the weight of coal consumed, and the total calorific power of the combustible obtained by proportion from the amounts of lead reduced by the given combustible, and by pure carbon.

According to Berthier pure carbon was capable of reducing thirty-four times its weight of lead*, and by multiplying the weight of lead reduced by the calorific value of pure carbon divided by 34, the thermal value of the combustible matter in any given coal may be determined. Professor Johnson used 20 grains (about $1\frac{1}{3}$ grams) of powdered coal sample for his litharge tests. Mr. Emery has suggested that this test of Berthier for determination of the heating power of coals by reason of its simplicity and readiness of application might be revived to advantage.†

Professor Johnson made many deductions from his chemical notes to show that the percentage of fixed carbon in the combustible was a direct measure of the heating power, and that, as the ratio of carbon to volatile matter diminished, the calorific power of the coal also diminished. While this is generally true, it is now well known that the anthracite coals, high in fixed carbon and low in volatile matter, are not possessed of the highest heating powers, nor have they shown the high efficiency of the smokeless West Virginia or the Cumberland coals when submitted to physical tests.

Ibid., p. 106.

† *Loc. cit.*, p. 30.

TABLE No. 22.—Cumberland Coals.

Source.	B. T. U. per pound of combustible.	How determined.	Authority.
George's Creek.....	16 048	Bomb Calorimeter.....	Hale and Williams. 1899.
"	15 958	"	" "
"	15 799	Analysis.....	" "
" (maximum)	15 141	Thompson Calorimeter.	Barrus, 1893.
" (minimum).	14 085	"	"
N. Y. & M. Mining Co.....	13 071	Litharge Test.....	Johnson, 1843.
"Coal in Store" (Mine)....	14 089	"	"
Atkinson and Templeman.	12 954	"	"
Easby and Smith.....	13 021	"	"
John Neff	13 237	"	"
D. & S. Coal Co.....	13 395	"	"

Upon the other hand, by the litharge test, Professor Johnson found for Lackawanna anthracite coals 14 432 B. T. U. per pound of combustible, which is much nearer some of the late determinations than the results obtained by this method for the Cumberland coals. For thirteen samples of anthracite coal tested by Professor Carpenter*, he found with his calorimeter for Drifton coal 14 480 B. T. U. per pound of combustible, and from eleven samples of anthracite coal tested by Mr. Barrus,† he found for an unnamed coal 14 508 B. T. U. per pound of combustible, results quite near that obtained for Lackawanna coal by the Berthier test fifty-six years ago by Professor Johnson. Parallel coals not having been tested during the earlier and later trials, better comparisons cannot be made, but from the limited data which are at all comparable, the suggestion arises that the Berthier method may be defective when applied to coals high in volatile matter, possibly by reason of incomplete or imperfect combustion of the hydro-carbons. When applied to carbons, and coals high in carbon, it seems that the method may give reliable results.

Physical Efficiencies of the Coals.

The relation of the heat accounted for in the steam to the total heating powers of the several coals tested is given in Table No. 23. The difference between this fraction and unity, represents the losses of heat by chimney draft, evaporation of moisture in the atmosphere and coal, rendered latent in the gases of combustion and the heat

* *Loc. cit.*, p. 17.
† *Loc. cit.*, p. 7.

losses by radiation and conduction. Notes sufficient to divide up the gross loss into the detailed losses were not taken, nor with the object which these tests had in view was it necessary, because the boiler, furnace and chimney conditions were alike for all coals. Exceptions should be made of the moisture in the atmosphere, for which neither coal nor boilers should be held responsible; but this in the author's previous experience has never exceeded $1\frac{1}{2}\%$ of the total heat of combustion.

Proximate Analyses of Coals.

The notes on the proximate analyses of these coals (Table No. 24) are means of three to five determinations for each sample. In these tests the sample of powdered coal was carefully dried at 105° Cent. for one hour, and then re-weighed for loss by moisture. The first few samples were then returned to the gas oven for fifteen or twenty minutes to ascertain if the additional exposure to the dry heat produced further loss of weight due to moisture. The early tests demonstrated that one hour was sufficient to expel the moisture from the samples, and all the after determinations were made upon this time of exposure to dry heat. The sample in the covered porcelain crucible was then exposed to the naked flame of a Bunsen burner until it ceased to give off gas at the edge of the cover. The crucible, without cooling, was then held over a blast lamp at white heat for three minutes, to expel the small amount of volatile matter which remained after the gas flame was extinguished. The crucible and contents were then cooled in a desiccator and weighed for the loss of weight of sample by volatile matter. The fixed carbon remaining in the crucible was then burned to constant weight in a current of oxygen over a Bunsen burner, after which operation the crucible was again cooled in the desiccator and weighed for loss due to the fixed carbon. The difference between the last weighing and weight of crucible represented the ash or non-combustible in the sample. No determinations for sulphur in the combustible or phosphorus in the ash were made, partly because of lack of significance of these elements in steam coals, and partly for lack of time, the city officials desiring the *résumé* of the work at an early date to use in connection with the annual contract for city coals.

The samples for proximate analysis and for the calorimetric deter-

TABLE No. 23.—Efficiencies Obtained From Various Coals.

Test No.	Coal sample.	Efficiency on Coal.	Efficiency on Combustible.
	Pennsylvania Coals.		
13....	Pittsburg. Brown. Second Pool. Lump	0.6500	0.6496
15....	" " " Barge Run	0.6542	0.6477
14....	" " " Mine Run	0.6081	0.6014
16....	" " " Nut and Slack	0.6091	0.6207
5....	" Catsburg. Third Pool. Lump	0.6629	0.6658
6....	" " " Barge Run	0.6315	0.6382
7....	" " " Mine Run	0.6102	0.6318
8....	" " " Nut and Slack	0.5910	0.6169
17....	" Cincinnati " Mine Run	0.6325	0.6557
18....	" " " Nut and Slack	0.6324	0.6451
9....	" Woods. Fourth Pool. Lump	0.6387	0.6557
10....	" " " Barge Run	0.6180	0.6496
11....	" " " Mine Run	0.6480	0.6731
12....	" " " Nut and Slack	0.6131	0.6496
	West Virginia Coals.		
35....	New River. Lump	0.6619	0.6766
49....	" Lump	0.6981	0.6218
1....	" Mine Run	0.6444	0.6606
31....	" Mine Run	0.6272	0.6734
27....	Loup Creek. Lump	0.6589	0.6698
20....	" Mine Run	0.5650	0.5751
21....	" Slack	0.5661	0.5806
26....	Pocahontas. Lump	0.6357	0.6627
22....	" Mine Run	0.6015	0.6333
2....	Kanawha. Steam Lump	0.5942	0.6136
3....	" Gas Lump	0.5519	0.5765
32....	Acme. Mine Run	0.6083	0.6311
33....	" Nut and Slack	0.5768	0.6156
34....	Keystone. Mine Run	0.6162	0.6458
23....	Winifrede. Lump	0.6306	0.6714
24....	" Mine Run	0.5851	0.5969
25....	" Nut and Slack	0.5710	0.5937
37....	Thacker. Lump	0.6294	0.6217
38....	" Mine Run	0.6246	0.6208
50....	" Mine Run	0.6332	0.6291
30....	" Nut and Slack	0.6013	0.6286
42....	Logan Consolidated Thacker. Mine Run	0.6210	0.6327
43....	Maritime Thacker. Mine Run	0.6335	0.6230
29....	Campbell's Creek. Mine Run	0.6079	0.6309
19....	" Nut and Slack	0.5881	0.5976
36....	Monongah. Mine Run	0.6062	0.6162
51....	Eagle. Mine Run	0.6228	0.6302
52....	Cedar Grove. Mine Run	0.6165	0.6164
53....	Mt. Carbon. Mine Run	0.6171	0.6267
54....	Chesapeake. Mine Run	0.6370	0.6306
55....	Coalburg. Mine Run	0.6198	0.6187
28....	Belmont Splint. Mine Run	0.6513	0.6949
39....	" Nut and Slack	0.6081	0.6734
	Ohio Coals.		
41....	Luhrig. Washed Egg	0.5629	0.5655
43....	" " Slack	0.5428	0.5159
44....	Wellston Shaft. Mine Run	0.6220	0.5895
45....	" Nut and Slack	0.5847	0.5450
	Kentucky Coals.		
47....	Jellico. Mine Run	0.6084	0.6123
40....	" Nut and Slack	0.5192	0.5645
	Tennessee Coal.		
48....	Jellico. Mine Run	0.6088	0.6081

minations were obtained by the customary method of quartering from several pounds to the small weights finally used. It was believed that all the samples excepting the fresh samples of New River (1), Pocahontas (26), Loup Creek (27), Eagle (51), Cedar Grove (52), Mt. Carbon (53), Chesapeake (54) and Coalburg (55), had lost in volatile matter during the interval of time which elapsed between the collection of the sample from the lot of test coal fired under the boilers, and the tests for moisture and combustible matter, and for heating power; but careful consideration of the proximate analyses of the old and new samples of New River, Loup Creek and Pocahontas coals, suggest that this may not generally have been true.

TABLE No. 24.—Proximate Analyses of Coals Tested at the Hunt Street Pumping Station.

Sample of coal.	Specific gravity.	Moisture.	Volatile matter.	Fixed carbon.	Ash.
Pennsylvania Coals.					
Pittsburg, Brown, Second Pool	1.317	1.17	26.93	68.15	3.75
" Catsburg, Third Pool	1.291	0.76	33.04	60.70	5.50
" Cincinnati, Third Pool	1.295	0.98	33.37	61.60	4.05
" Woods, Fourth Pool	1.292	0.65	31.95	61.80	5.60
West Virginia Coals.					
New River (old sample)	1.270	0.50	17.85	80.20	1.45
" (new sample)	1.270	0.90	18.60	78.10	2.40
Loup Creek (old sample)	1.275	0.55	19.25	78.05	2.15
" (new sample)	1.275	1.15	19.15	78.75	0.95
Pocahontas (old sample)	1.277	0.45	14.95	80.60	4.00
" (new sample)	1.277	0.52	18.80	79.43	1.25
Kanawha, Steam	1.277	0.80	31.28	65.82	2.10
" Gas	1.261	0.95	37.35	57.95	3.75
Acme	1.273	1.00	31.50	65.40	2.10
Keystone	1.306	0.78	28.92	62.00	8.30
Winifrede	1.297	1.05	32.40	63.70	2.85
Thacker	1.324	0.73	31.32	62.95	5.00
Logan Consolidated Thacker	1.309	0.70	32.60	62.85	3.85
Maritime Thacker	1.301	0.68	31.67	63.05	4.60
Campbell's Creek	1.278	0.95	38.00	58.15	2.90
Monongah	1.282	1.05	33.15	62.50	3.30
Eagle	1.268	1.00	30.53	65.87	2.60
Cedar Grove	1.308	0.92	35.35	60.26	3.47
Mt. Carbon	1.266	0.73	31.87	63.07	4.33
Chesapeake	1.274	1.15	34.28	61.30	3.27
Coalburg	1.289	0.90	38.17	56.00	4.93
Belmont Splint	1.269	1.28	34.22	63.30	1.20
Ohio Coals.					
Luhrig, Washed	1.306	2.87	33.47	56.76	6.90
Wellston, Shaft	1.293	3.38	34.07	55.25	7.30
Kentucky Coal.					
Jellico	1.273	1.68	36.62	60.85	0.85
Tennessee Coal.					
Jellico	1.275	1.80	36.10	61.00	1.10

(The above determinations were made by Mr. Henry C. Hill.)

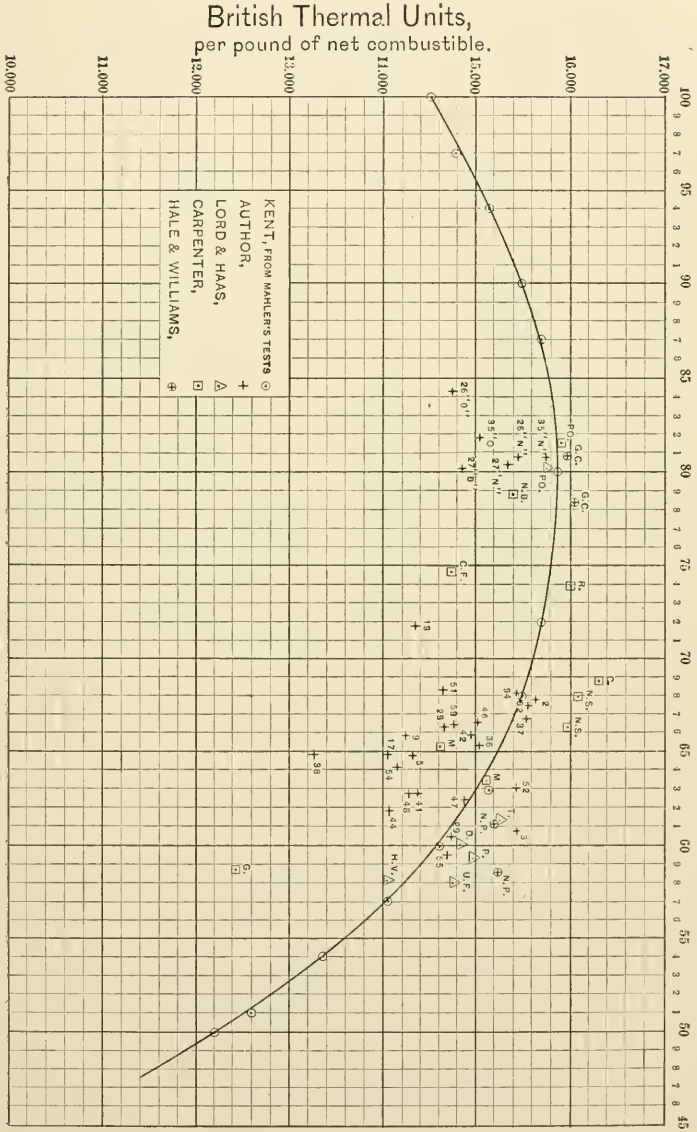

British Thermal Units,
per pound of net combustible.
Percentage of Fixed Carbon in the Net Combustible.
KENT, FROM MAHLER'S TESTS
AUTHOR,
LORD & HAAS,
CARPENTER,
HALE & WILLIAMS,

Fig. 7.

Kent's Estimate of Total Heat of Coals from Proximate Analysis.

Mr. William Kent has given a table of the proximate heating values of coals deduced from Mahler's experimental data.* In this table he attempts to show within limits of error of 3% the B. T. U. of coals, having previously determined percentages of fixed carbon of the net combustible. The figures in this table have been plotted as co-ordinates on the diagram (Fig. 7) and connected by a smooth curve. Upon the diagram the heat values obtained by the author with the Carpenter calorimeter, together with the results of tests by Professor Carpenter, Lord and Haas, and Hale and Williams, have also been plotted for comparison with Mr. Kent's curve. An examination of this diagram indicates a wide difference between the heating values of certain coals as given by the curve and from calorimetric tests, and this doubtless is due to the variable nature of the volatile matter.

From any series of experiments on bituminous coals like those by Mahler, curves can be obtained which will approximately fit the present conditions, but not be generally applicable to another series of tests of coals from other localities. By reference to Table No. 25 it will be seen that four of the best coals calorimetrically tested, viz: Thacker, Acme, Kanawha Steam and Kanawha Gas coals, having heating powers of 15 554, 15 589, 15 653 and 15 440 B. T. U. per pound of net combustible, contained scarcely two-thirds of fixed carbon, and by the Kent method should have rated in heating power considerably lower than the smokeless coals; when, in fact, by the calorimetric method these coals were not inferior to the coals giving the highest heating powers and evaporative results.

Referring again to the table it will be noticed, that as a rule, the actual heating power of the net combustible† was considerably under the value which would be assigned to it by the Kent curve. It is scarcely probable that the heats obtained by the author were too low, because combustion must have been perfect in the presence of pure oxygen under pressure, and as shown by the absence of smoke and soot in the combustion chamber, and at the orifice of exit for the waste gases; and was complete as shown by the fair agreement of the ash by calorimetric method and proximate analysis.

* Kent's "Mechanical Engineers' Pocketbook," First Edition, p. 634.

† Coal—(ash + moisture).

TABLE No. 25.—Heating Values of Net Combustible.

Coal.	Percentage of fixed carbon in net combustible.	B. T. U. Per Pound of Net Combustible.	
		By calorimeter.	By Kent's curve.
Pennsylvania Coals.			
Pittsburg, Brown, Second Pool	71.70	14 381	15 670
" Catsburg, Third Pool	64.75	14 317	15 260
" Cincinnati, " "	64.86	14 036	15 270
" Woods, Fourth "	65.93	14 213	15 350
West Virginia Coals.			
New River (old sample)	81.80	15 028	15 815
" (new sample)	80.77	15 757	15 840
Loup Creek (old sample)	80.22	14 837	15 850
" (new sample)	80.44	15 336	15 845
Pocahontas (old sample)	84.35	14 738	15 770
" (new sample)	80.86	15 456	15 840
Kanawha Steam	67.79	15 653	15 470
" Gas	60.81	15 440	14 780
Acme	67.49	15 589	15 460
Keystone	68.19	15 448	15 490
Winifrede	66.29	14 693	15 375
Thacker	66.78	15 554	15 400
Logan Consolidated Thacker	65.85	14 968	15 345
Maritime Thacker	66.56	15 008	15 380
Campbell's Creek	60.48	14 789	14 690
Monongah	65.34	15 022	15 300
Eagle	68.33	14 625	15 500
Cedar Grove	63.03	15 422	15 120
Mount Carbon	66.43	14 761	15 380
Chesapeake	64.13	14 086	15 210
Coalburg	59.47	14 686	14 510
Belmont Splint	64.91	13 262	15 275
Ohio Coals.			
Luhrig, Washed	62.92	14 357	15 105
Wellston Shaft	61.86	14 080	14 960
Kentucky Coal.			
Jellico	62.43	14 880	15 030
Tennessee Coal.			
Jellico	62.82	14 299	15 100

One serious objection to the general application of Mr. Kent's curve is the variable nature of the volatile matter in bituminous coals, which may have an effect on the calorific power. Thus, with constant fixed carbon, when the hydrogen is relatively high and the oxygen relatively low, the heating power of the volatile matter will be high, and when the hydrogen is relatively low and the oxygen relatively high, the heating power of the volatile matter will be low. (Varying amounts of sulphur in the coal will have the effect of slightly modifying this.)

Table No. 26, from analyses by Professors Lord and Haas,* will show the influence of varying relative amounts of hydrogen and oxygen on the heating power of coals:

TABLE No. 26.—Relation of Hydrogen, Oxygen and Heating Power of Coals.

Test No.	Coal.	Percentages.				Heating Power.	
		Ash.	Fixed carbon.	Hydrogen.	Oxygen.	By calorimeter. Calories.†	By analysis. Calories.†
24	Freeport.....	9.17	73.15	4.98	7.41	7 464	7 393
30	"	9.10	71.40	4.62	10.68	7 113	6 970
16	Pittsburg.....	7.95	76.56	5.22	7.00	7 630	7 719
34	"	9.05	73.91	5.15	8.89	7 354	7 404
32	Darlington.....	8.70	72.78	4.93	10.57	7 245	7 173
33	"	8.80	72.82	5.25	8.55	7 304	7 395
6	Hocking Valley.	5.83	69.42	5.35	16.27	6 882	6 790
8	" " .	9.67	66.59	5.16	15.57	6 494	6 520
11	Thacker.....	6.50	78.90	5.14	6.88	7 768	7 876
14	"	6.05	78.40	5.19	7.56	7 867	7 831
3	Pocahontas.....	5.63	85.46	4.25	3.24	8 185	8 246
5	"	4.80	85.40	4.39	3.94	8 280	8 258

† Pound calories × 1.8 = B. T. U.

Considering the Freeport, Pittsburg and Darlington coals, the relation of hydrogen and oxygen varies sufficiently to overcome the modifying influence of the differences in percentages of fixed carbon. With the Hocking Valley coal the relation of the hydrogen and oxygen is fairly constant, but the combustible varies in heating power by reason of the variable proportion of fixed carbon, while with the Thacker and Pocahontas coals all conditions of the combustible are fairly constant, and the heating powers likewise are constant.

The data in Table No. 27, from ten samples of bituminous coals, are taken from Professor Carpenter's paper, descriptive of his form of coal calorimeter,‡ and compared on the heating power of the net combustible with the heats obtained for the given percentages of free carbon by Mr. Kent's curve.

* *Loc. cit.*

‡ *Loc. cit.*

TABLE No. 27.

Source of coal.	Percentage of free carbon.	B. T. U. Per Pound of Net Combustible.	
		By calorimeter.	By Kent's curve.
Nova Scotia	67.93	16 093	15 475
" "	66.27	15 964	15 370
Pennsylvania	65.19	14 601	15 290
"	73.93	16 000	15 750
"	68.78	16 315	15 510
"	63.47	15 106	15 160
"	74.73	14 753	15 780
"	78.87	15 396	15 835
West Virginia	81.47	15 872	15 820
Illinois	58.65	12 435	14 400

Specific Gravity of the Coals.

The specific gravity given in the table of analyses of each kind of coal—a mean of five or six determinations—will be found of value in estimating the stowage for marine, railway and factory uses where the coal storage space is limited. Thus a coal of given specific gravity and heating power will occupy less bulk for a given steam effect than will another coal lower in specific gravity and of equal heating power. The relative values where stowage is a consideration will be approximately represented by the products of the heating powers and specific gravities of the several coals, and to illustrate the effect of this, the following comparisons of a few coals have been made:

TABLE No. 28.

Coal sample.	Product. heating power (B. T. U.). and specific gravity.	Relative stowage value.
Pittsburg (average)	17 339	89.70
New River	19 257	99.63
Loup Creek	19 093	98.78
Pocahontas	19 325	100.00
Acme	19 329	100.00
Thacker	19 232	99.50
Eagle	17 803	92.10
Cedar Grove	19 186	99.26
Luhrig, Washed	16 926	87.57
Wellston Shaft	16 170	83.66

Considering that the New River, Loup Creek, Pocahontas and Thacker coals, in addition to their other good qualities, are all so-

called smokeless coals, the importance of these for naval and railway purposes is quite apparent.

It should be matter for congratulation by the people of the United States residing east of the Mississippi River, that Nature has been so bountiful in locating such vast tracts of high-grade steam coals as are found in the States of Pennsylvania, West Virginia and Kentucky, especially as these coals are easily mined and cheaply transported to the seaboard, or to points of consumption inland. Some of these coals, like Pocahontas, New River, and Acme, are not surpassed for steam use by any known coals in the world, and when boilers and furnaces have been devised to obtain the highest practical efficiencies, then these coals will be made to contribute in a higher degree to the commercial prosperity of the country.

Reference to the physical efficiencies of these coals (Table No. 23), indicates that none of the coals was giving very high results, and some, undoubtedly, were worked at a disadvantage, although the evidences of this were not apparent at the time; but high efficiency of the low-grade coals cannot always be attained except in furnaces especially adapted for them. All the losses of heat by chimney draft, etc., should be limited to 30% of the ultimate heat value of the coal; and in designing boilers and furnaces, due regard should be had for the work to be performed and the coal to be used. But variations in any large degree in the amount of work, and the kind and grade of coal burned, will in most instances be attended with reduced efficiency of the fuel.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE ARTIFICIAL PRESERVATION OF RAILROAD TIES BY THE USE OF ZINC CHLORIDE.

By WALTER W. CURTIS, M. Am. Soc. C. E.

TO BE PRESENTED MAY 17TH, 1899.

Over a year ago the author became interested in the question of the present commercial value of the zinc-chloride process for preserving timber, and particularly ties; with the result that he began an investigation to determine for himself the value of the treatment and the occasion for it in the United States.

Since the report made to the Society by its Committee* in 1885, there has been but little published on the subject, and that has been in pamphlet form, making it extremely difficult, if indeed possible, to secure recent and accurate data on the subject. Probably many will be surprised to learn that during the last 12 years something like 10 000 000 cross-ties have been thus treated, and that during the present year there will probably be 1 500 000 ties treated.

It is hoped that this paper may to some extent add to the knowledge on the subject and place in accessible form that which has already existed for those who knew where to find it.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in *Transactions*.

* *Transactions*, Am. Soc. C. E., Vol. xiv, p. 247.

Ties in General.

Any investigation of the timber resources of the country is bound to be exceedingly disappointing, on account of the impossibility of securing any complete, or even correct partial data.

The Government, through the Department of Forestry, has investigated the subject, and has collected much information as to the supply, the consumption, the characteristics of the material and the best methods of using and economizing American timbers. With reference to the special subject of railroad ties, the Department, in 1882 and 1883, by means of circulars, collected data from 283 different corporations, representing over 70 000 miles of track. This data covered the size and number of ties used per mile, the kinds of timber used, its durability and cost, with general information as to the sources of supply, etc. In 1886 inquiries covering somewhat similar ground, resulted in replies from 35 representative corporations operating 46 000 miles of track. The first indicated the average life of ties to be as follows: White and burr oak 7 years, cypress 9 years, redwood and cedar 11 years, and for other woods generally 5 years. In the later report it is stated:

"While no general rule can be laid down as to the lasting quality of timber imbedded under the track and used in bridges and trestle work, subject, as it is, to the endless varieties of conditions of soil and climate in different sections of the country, nevertheless, it may be safely assumed that 7 years is the limit of the average usefulness of ties of the best kinds of hard wood, and 4 years for the soft woods."

The 46 000 miles of road covered by the 1886 report, used in renewals, the preceding year, 16 700 000 ties, at an average cost of 35.6 cents each. "The average number of new ties required annually for each mile, to replace decayed ones, is 365; the average duration of ties, 7.2 years."

In the report on metal ties and the preservative processes for wooden ties, prepared by E. E. R. Tratman, Assoc. M. Am. Soc. C. E., under the direction of Mr. B. E. Fernow, Chief of the Division of Forestry, and issued in 1894 as Bulletin No. 9, the situation is summarized thus:

"Briefly recapitulating statistics as given in the previous bulletins, it may be stated, that with an average of 2 500 ties per mile, the 230 000 miles of track, including second, third, fourth and side tracks, represent 575 000 000 ties in service. Renewals require an-

nually upwards of 75 000 000 ties" (corresponding to $7\frac{2}{3}$ years of life) "new construction 13 000 000 ties, or the total annual consumption in round numbers of 90 000 000 ties, equal to 450 000 000 cu. ft. of forest-grown material, to which some 60 000 000 cu. ft. for bridges and trestle work must be added, making the total consumption for railway purposes exceed 500 000 000 cu. ft. This requires the annual culling of the best timber from probably more than 1 000 000 acres, and the annual product of at least 50 000 000 acres in good condition, or more than 10% of the present forest area of the United States reserved for this one use alone."

Benjamin Reece, M. Am. Soc. C. E., demonstrates* conclusively the very largely increased relative importance of tie renewals as compared with rail renewals; the expenditure for ties on various roads having increased relatively to rails in the years 1880 and 1890 nearly 300 per cent. Most of this remarkable increase was undoubtedly due to the decreased cost of rails per ton; and the data are lacking in Mr. Reece's paper to determine how much, if any, is properly attributable to increased cost per tie or to decreased life. The increased importance to the railroads of the expense of tie renewals, however, is not affected, whatever the reason; and the evidence of the urgent need of study and effort to reduce the cost of such renewals is sufficient.

It is difficult to learn much about the history of the ties on many American roads. A rail carries with it the name of its maker, the year made and a guaranteed life. The tie is simply a stick, looking much like any other, with no marks on it and no record anywhere. The officials know that the first ties were originally laid during a certain year; that perhaps these were largely replaced when new rails were put down at a later date; that some have been renewed each year since; and generally that is all that can be accurately stated. Some roads can do more, and can tell how many ties have been replaced on each section each year; but if an attempt is made to ascertain the average life of a certain kind of tie, or of all kinds in the main track, it will almost invariably result in disappointment. In certain instances there may be particular stretches of road where for special reasons the roadmaster remembers more definitely how many ties have been removed. This is generally where the ties have been remarkably durable, or else short-lived, and, based on that recollection, the life of that particular kind of tie is apt to be given as longer or shorter than a general average would justify.

* "The Increasing Cost of Railway Tie Renewals." *Transactions*, Am. Soc. C. E. Vol. xxvii, p. 640.

The trouble is that there has never been, nor is there now, any generally accepted method of marking ties; and indeed there is no particular advantage in knowing just how long any single untreated tie has been in place. Presumably the best obtainable of the kind are bought, and when they are destroyed more must be purchased. The kind selected has usually been the best obtainable from the adjacent land.

It is hoped that the discussion of this paper will bring out the records of careful determinations of the life of various woods used as ties.

It may be concluded that at present the only information regarding the probable life of ties, free from the very large element of personal opinion, to be secured from American roads generally, is to be obtained by taking the total tie renewals per mile of road per year, averaged over, say, 5-year periods. This will indicate the average number of annual renewals per mile, and whether the average life of the ties is varying. Covering sidings as well as main tracks, however, such information is misleading, as regards the life of a tie in the main line; must be carefully interpreted in the light of the particular conditions for each road, and can be considered as indicative only. To compare one road with another, the conditions must be alike; about the same relative proportion of siding and main track; of hard and soft wood, and of tie plates used; and the same climatic conditions must obtain. As indicative of the latter, J. F. Wallace, M. Am. Soc. C. E., Assistant Second Vice-President of the Illinois Central Railroad, states that white oak ties on his lines south of the Ohio River have an average life of from 4 to 6 years, depending upon the height of the land where the timber was grown, the season when cut and the kind of ballast.

North of the Ohio, in central and southern Illinois, the average is from 7 to 9 years, and in northern Illinois and Iowa it is from 9 to 12 years.

Through the kindness of the officers of some roads and from an examination of the annual reports of others, the author has secured certain data on renewals, which are of sufficient interest to warrant preservation. Mr. Wallace states:

"The average number of ties used each year on the Illinois Central Railroad for the past 18 years has been, approximately, 300 to the mile. The annual number used per mile, however, has varied from

as low as 221 to as high as 434. I estimate it will require an average of 350 ties per mile per annum, taking our line as a whole, to keep up normal repairs, on the basis of 3 000 ties per mile of track."

Generally speaking, white oak ties are used north of Memphis, and cypress ties south thereof.

E. A. Handy,* M. Am. Soc. C. E., has given the annual renewals per mile for his road for the past 15 years, showing a variation from 174 to 359 per mile. The average per year for the 5 years from 1883 to 1887 is 310; for the 5 years from 1888 to 1892, 317; from 1893 to 1897, 209. The only explanation obtainable for this very remarkable decrease is the reported construction of a large amount of second track in 1892, 1893 and 1894, resulting in increased mileage without increased tie renewals. However, as the total trackage of all kinds in 1891 was 2 512 miles, and in 1897 was 2 651 miles (an increase between these years of less than 6%), the one-third decrease in renewals is hardly accounted for. The extensive preparations made prior to 1893 for the demands of traffic during the Exposition at Chicago may have had something to do with it. As evidence that it was not due solely to extreme economy during the poor years just past it may be stated that the renewals for 1897, while in excess of the four previous years, were only 245 per mile. White oak ties are used principally.

The annual reports of the Chicago and Northwestern Railroad give the average annual renewals for the years 1883 to 1887 as 206 (not exact, as the length of side-tracks for 4 years was estimated), 1888 to 1892, 216; 1893 to 1897, 207. During these years the track mileage increased from 4 100 to 7 057, the increases being made by yearly additions of reasonable uniformity. So far as an indication of the real average life of ties is concerned, the figures are deceptive.

An average renewal of 210 ties corresponds to a life of $13\frac{1}{3}$ years, assuming 2 800 ties to the mile, and, on the mileage of 1897, to a total number of ties renewed of 1 481 970. If the 13 years in fact represented the average life, and no allowance was necessary for the ties destroyed earlier on the extensions before the average life had been reached, this number of renewals should correspond approximately to that on the mileage in 1884, which was about 4 400. As this gives a renewal per mile of 337 ties instead of the actual 210, it is evident that this latter figure is deceptive and is due to the rapid increase in the length of the road.

* Chief Engineer of the Lake Shore and Michigan Southern Railroad.

Further trial computations indicate to the author that the probable average life is about 10 years, corresponding to an average renewal rate of about 280 ties per year.

Mr. J. E. Blunt, Chief Engineer, states that the ties in all their principal main lines are white oak, now of a decidedly poorer quality than formerly used, from which they get 8 to 9 years' life. The balance of their ties are cedar, giving 12 to 14 years' service. The ties, per mile, average about 2 700.

R. Montfort, M. Am. Soc. C. E., has kindly furnished statistics for the Louisville and Nashville Railroad, from which the following results have been deduced:

Average renewals per mile for 15 years prior to 1898.....	360
" " " " 10 " " "	342
" " " " 7 " " "	334
" " " for the year 1896–97.....	398
" " " " " 1897–98 as recommended.	375

Noting that the figures for the renewals for the 10 and 15-year periods include only those divisions which have been built that length of time, on which the amount of new track laid is presumably quite small, it is fair to assume the average life of ties on this road to be about 8 years.

On the Chicago, Milwaukee and St. Paul Railroad the record, prior to 1889, was not readily obtainable. Beginning with that year's annual report the renewals each year to 1897 were, 280, 210, 204, 228, 245, 245, 266, 262 and 242. The average of these is 243, corresponding to a life of about 11 years, the average ties in use per mile of all tracks being about 2 700. During these years the mileage increased from 6 810 to 7 747, an amount probably not sufficient to greatly affect the apparent life of the ties. At present about 50% of new ties purchased are white oak, and about 45% white cedar. In past years a considerably larger percentage has been white oak.

Through Thomas Rodd, M. Am. Soc. C. E., the statistics for the Pennsylvania Railroad, West of Pittsburg, have been furnished by Mr. Charles Watts, General Superintendent. The average life of various woods used for ties is:

Oak, 9.2 years; hackberry, 4 years; tamarack, 5 years; cedar, wild cherry, honey locust and black walnut, 8 years.

The method of determining these periods is not stated. The tie renewals since 1883 have varied from 168 to 363. The average by 5-year intervals is: 1883 to 1887, 236; 1888 to 1892, 289; 1893 to 1897, 212. During these years the cost of ties has increased about 10%, and the quality has deteriorated. The length of haul in the woods has increased an average of 6 miles for the system, indicating the exhaustion of the nearer lands.

Mr. J. B. Berry furnished data for the Fremont, Elkhorn and Missouri Valley Railroad, of which road he was Chief Engineer until quite recently.

The figures do not extend back beyond 1893, but they are remarkable, both for their extreme smallness and the excellent system indicated. The entire renewals for 1 344 miles of main track, none built later than 1888, and some in 1868, were 470 per mile for the years 1893 to 1898, inclusive, or less than 80 per mile per year. The renewals figured for 1898 were those recommended to be used. This very small number is due to various causes : The location of the line in an extremely dry section, the rigid economy required during the period of commercial depression, the light traffic and the use of large numbers of cedar ties under their most favorable conditions. A considerable part, however, is undoubtedly due to Mr. Berry's systematic and constant inspection of the ties, as requisitioned for and when removed from the track as destroyed. Each year tables similar to those shown in Forms A and B have been prepared, giving the data for every portion of the road.

FORM A.

Location.	Miles of main track.	Year road built.	Average number of ties per mile of main track.	Assumed average life of ties in years.	Making average per mile per year for main track.	Number ties requested per mile for 1898 main track.

FORM B.

Location.	Miles of main track.	Year road built.	Renewals per Mile.					Proposed for 1898.
			1893.	1894.	1895.	1896.	1897.	

All ties removed during the year are piled up and not destroyed until inspected by the Chief Engineer, and to this *post mortem* Mr. Berry attributes most of the credit. Presumably he claims no originality for either the method of tabulating the facts or the final inspection. Mr. Reece, in the paper previously referred to,* states that he inaugurated the same system on the Lake Shore and Michigan Southern Railroad with great benefit; but as the author has found sufficient evidence of the need of such systematic care, he feels justified in presenting Mr. Berry's method to the Society.

None of the roads mentioned has used treated ties more than experimentally, if at all. The figures given for renewals, excepting in the last instance, are per mile of all tracks, both main and sidings, and consequently correspond to a longer life than is obtained in the main track. This accounts for the discrepancy between the life of ties given by the average renewals, and that previously stated by the various roads, as the actual life of particular woods in the main track service. The average annual renewals for the past 15 years, indicating very clearly the climatic effect, are:

Pennsylvania Railroad, West of Pittsburg.....245;
Lake Shore and Michigan Southern.....280;
Chicago and North Western .....280 (corrected for extensions);
Chicago, Milwaukee and St. Paul.....243 (for 9 years only).
Illinois Central.....300 (350 on basis of 3 000 ties per mile);
Louisville and Nashville..... ...360.

Ties perish by the mechanical destruction of the fibers under the rail; splitting by seasoning and cutting out of fiber by respiking; and by natural decay. In its normal condition, to resist these three influences, a timber is required which shall have sufficient hardness to resist the mechanical injury; a high degree of coherence between fibers ; and a sap of such chemical composition as will offer the least encouragement to the development of the fungoid life which destroys the fiber.

The destruction of the fibers under the rail is generally attributed to the cutting and abrading action due to the deflection of the rail and

* *Transactions*, Am. Soc. C. E., Vol. xxvii, p. 640.

tie and the longitudinal movement of the rail as the load passes over it; and the direct compression of the timber is considered as too small to affect it. If a 60-lb. rail has a bearing on the tie of $4\frac{1}{4}$ x 8 ins., or 34 sq. ins., an engine with 30 000 lbs. on an axle would cause a pressure of 441 lbs. per square inch on the tie, making no allowance for distribution by the rail. According to investigations of the Forestry Department of the Government, the pines and cedars, when saturated with water, as the surface of a tie is likely to be at times, have a resistance to a 3% penetration across the grain, of from 400 to 500 lbs., corresponding to the assumed working limit of compression. As the condition of a full bearing on a new tie is decidedly improbable, it is by no means impossible that in such woods, the initial cutting under the rail may be due largely to the direct compression.

The injury by splitting from seasoning can be reduced by covering with ballast a larger part of the ties subject to such action, and that from respiking by carefully plugging all spike holes. As to the general practice regarding plugging, the author is not qualified to speak. He understands that it is generally neglected, but he knows of one road which supplies the section men with turned plugs of seasoned elm; and the instructions are that every hole shall be carefully plugged, whether the spike is redriven into it or not, unless the tie is so decayed as to necessitate removal within a year. The results are reported as being very satisfactory and the slight expense of providing the proper material for such use fully justified.

To secure the best results with any tie, it is necessary that the tree be cut at the proper time, the wood seasoned before being placed in the track, and the roadbed well drained. The necessity for cutting at a particular time has been attributed to various causes: The greater amount of sap, the greater activity of the sap, the change of position of the sap in the tree and the change in the character of the sap at certain seasons, which were consequently to be avoided for cutting. The latest dictum on the subject, as contained in the 10th Bulletin of the United States Forestry Division, and reprinted in Professor J. B. Johnson's "Materials of Construction," is:

"Wood cut in the fall is more durable than that cut in summer, only because the low temperature of the winter season prevents the attack of the fungi, and the wood is thus given a fair chance to dry. Usually, summer-felled wood, on account of the prevalent high

temperature and exposure to sun, checks more than winter-felled wood; and since all season-checks favor the entrance of both moisture and fungus, they facilitate destruction."

Whatever the reason, there seems to be no question but that in the northern part of the United States, timber should be cut in the late fall and winter. The value of seasoning seems also to be undisputed, but the record of experience on the subject is small. That the roadbed should be well drained, to prevent the decay of the ties is evident; and that the ballast should not cover the ends of the ties so as to prevent the ready escape of water and the circulation of air around them, is good practice which is frequently ignored.

The immediate cause of the removal of the average tie is dependent upon the character of the timber used, the climatic conditions, the severity of tie inspection, both when purchased and after service in the track, the character of the roadbed and the amount of traffic. It may be decay or the mechanical destruction under the rail. If it is decay, it may be the general failure of the stick, or local rot under the rail. In any case, it may be concluded that anything which will postpone the decay of the wood, will add to the life of the tie, for where the fibers are injured, rot will find favorable conditions for development, and where the rot has begun, the fibers are reduced in strength and quickly destroyed. With certain woods durability can be secured by mechanical means for preventing the breaking down of the fibers under the rail, as by tie plates. This is especially true of cedar, cypress and redwood. Woods which are of such texture that they resist decay and mechanical injury equally well for considerable periods, and have consequently been the most satisfactory timber for ties, are unfortunately few; being the white, burr, chestnut, post and rock oaks. With most of the American woods, however, nothing will materially prolong the life of the tie which does not essentially change the condition of the sap. Such change may in itself be sufficient to make a timber perfectly adapted for use as a tie, or it may still be necessary to protect it from mechanical injury; the determining factors being the character of the timber operated on or the process used.

The oaks which have been in the past and still are in some localities the principal source of the best ties are rapidly being exhausted. It is an especially favored road that can to-day secure any considerable

supply of good white or burr oak from its own territory, and even then it is the policy to use the oak for but a part of the supply, supplementing it with other and less desirable materials. The report of one of the committees of the Roadmasters' Association, made to the annual meeting held in Denver in September, 1898, may be quoted as indicating the general conviction that the day of the hardwood tie is nearing its close. This report recommended:

"That tie plates with soft ties be used in preference to hard ties without any plate, when the natural life of the soft tie is equal to or greater than the natural life of the hard tie, cost being equal."

The cedar is being more largely used with and without tie plates, but any largely increased demand for cedar, due to its adoption generally, would probably cause a rapid increase in cost. It is not a wood that is generally or uniformly distributed throughout the country; and even in Chicago, closely adjacent to the cedar supplies, the increased demand of the last 3 years has advanced the price from 25 cents to 38 cents.

The future supply of ties must undoubtedly come from what are now considered the second-class or even waste woods. These, after chemical treatment, will resist decay for from 10 to 12 years and will generally cost less than the white or burr oaks.

One of the causes which has prevented the more general adoption of preservative methods, has been the low cost of timber, even of the best quality. Notwithstanding the generally admitted fact that the supply of such timber is materially less than it was 10 years ago, in many localities the cost is no greater than it was then; and in some cases it is even less. One of the Western roads which 10 years ago paid 50 cents each for the white oak ties purchased along its own lines, is now paying for the same tie 45 cents; and owing to stricter and more thorough inspection, it is getting better timber. However, although this road runs through a magnificent timber country, and purchases all the oak offered for sale on the line, it has been found necessary to supplement the supply by importations from the South at higher prices, and by the increased use of soft woods. This year a larger percentage of soft woods will be used than has been the custom.

One explanation of this shortening of the supply without a corresponding increase in cost may perhaps be that given in one of the Government reports on the subject:

"The agencies employed in supplying the enormous quantities of ties required by the railway system may determine, to a great extent, the amount and price of material offered in the market. In many parts of the country the cutting and hauling of ties to railway stations is performed mostly by farmers to whom this labor is a source of ready cash and employment of spare time. Owing to this mode of supply, an abundance of material is steadily offered, giving railways full control in the selection of the best quality and in the price paid; while by a regulation of the rates of freight they can prevent an undue export to the treeless sections of the country, or check materially the lumber industry of certain localities. In this way prices are kept down and a show of seeming abundance is maintained which does not in the least prove the supply to be adequate, but merely shows that the great number of people engaged in this industry are willing to haul ties to the railway over distances steadily increasing as the timber nearest the lines is cut off. Immense quantities of tie-timber are thus cut down and delivered to the railways at prices below its real value. At the time the leading lines west of the Missouri River were constructed, extensive tracts of white oak timber were laid waste in the Western States. And the same process is going on throughout the white cedar regions of the North, from which immense quantities of ties are thrown on the timber market. These channels of supply, based solely on considerations of monetary profit, will continue to bring to market the forest wealth of many sections of the land, but in many instances and at a time not far distant Nature's bountiful supply will be exhausted, and the reckless system of forest clearing will of necessity be a thing of the past."

This was written 12 years ago and the truth of it is evidenced by the greater scarcity, and, so far as many roads are concerned, practical exhaustion, of the white oak supplies in the northern part of the country.

The constant extensions of railroads into new territory, opening up forests before inaccessible, has had even more to do with the stationary prices. A recent number of a trade publication* says:

"So long as railroad rails rest on wood, so long the white oak tie will be wanted above any other available wood. A large proportion of the mileage of western roads has for many years been laid with this tie and on ties brought from the woods of the three white pine States. But the time has now come when this source of supply is failing rapidly.

"The States of Michigan, Wisconsin and Minnesota to-day furnish probably one white oak tie where 5 years ago they furnished five. With the present rate of cutting these ties in these States, the supply 5

* *The Mississippi Valley Lumberman.*

years hence promises to be insignificant. Four-fifths of the white oak ties now being bought by the roads running through these States are brought from the South. In time the supply will be almost wholly from the South.

"Though the supply of white oak suitable for ties has grown so small in the North, the market value of this tie has remained almost stationary because of the recent influx of the Southern tie. The Omaha, the Soo, the Wisconsin Central and the Milwaukee lines have this year drawn the bulk of their white oak from the South. A reduction in freight rates is in part responsible for this."

Experience with Southern oaks when brought north seems to be somewhat contradictory, but is probably unsatisfactory, while the cost is high. As a consequence, it is believed that the supply of ties generally and very shortly will have to come from the inferior grades of timber, with a corresponding large increase in the number of annual tie renewals, unless it is possible to so treat them at a low cost as to give them a life equal or superior to that now secured from the oaks.

Résumé of Report of 1885.

In 1885 a Committee of this Society, consisting of nine members, presented an exhaustive report on the subject of the preservation of timber; which, with the discussion thereon, participated in by sixteen other members, still stands as the authoritative statement on the subject; and many of the general conclusions of the Committee remain uncontroverted to-day. Very naturally, however, during the past 12 years much has been learned, some of the ideas formerly held have been modified and confirmatory evidence produced as to others.

The conclusions in the report on the preservation of ties were:

"If the exposure is to be that of a railroad tie, creosoting is, doubtless, the most perfect process to use, but in view of the expense, it may be preferable to use a cheaper process, dependent somewhat upon the location.

"With our present knowledge, and as a result of this investigation, we believe Burnettizing (the zinc chloride process) is the advisable process to use for ties at present in this country."

The facts upon which the conclusion as to the sufficiency of Burnettizing was based were the following:

A street railway in Cambridge, Mass., was laid with spruce stringers and sleepers in 1855. In 1883, 28 years later, the stringers

were all worn out, but the president reported that "many, and, I think, the majority of the sleepers are in good condition to-day."

On the Vermont Central Railroad ties were treated in 1856 to 1860, at which time the plant was abandoned and the entire matter lost sight of until 1879, "when an old side track was removed, which had not been in use for several years, and which was nearly covered with earth and grass; still the hemlock ties were then found to be nearly sound" after over 20 years.

"The reasons for abandoning the Burnettizing works upon this road would seem to be that the officers in charge at that time lost faith in the theory, and, as it was an experiment, they did not learn of its value until recently discovered in the manner referred to."

In a wharf and in a street railway in Boston, Mass., spruce plank and track stringers which were Burnettized in 1857 were sound in 1872, or 15 years afterward.

On the Chicago, Rock Island and Pacific Railroad eight spans of Howe-truss bridges built in 1860 were still in use and in fair condition in 1882. On this road 2 000 ties of hemlock, pine, tamarack and cedar were laid in 1866. In 1882 about 75% of the hemlock was still in the track and good for several years longer; the pine and cedar ties had all been removed sometime during the 15 years—the tamarack lasting about as long as the hemlock.

On the Boston and Albany Railroad a 100-ft. bridge, built in Boston by the late E. S. Philbrick, M. Am. Soc. C. E., was provided with green spruce Burnettized ties in 1860 which lasted 9 years. They were replaced by untreated pine, lasting 6 years, and these were replaced by untreated chestnut, lasting 7 years.

Untreated spruce is stated by Mr. Philbrick to have a life of 4 to 5 years.

On the Lehigh and Susquehanna Railroad, L. L. Buck, M. Am. Soc. C. E., reported in 1883 that he had examined Burnettized maple, beech and hemlock ties laid in 1867–68 which had "resisted decay almost perfectly. Most of the treated ties appeared good for 7 or 8 years longer."

This represents the American experience in 1885 favorable to zinc chloride as a tie preservative. There were, in addition, the results of 30 years of German experience, which can be summarized from Councillor Funk's paper in the report as follows: In 1880, out of 60 000 000

ties laid on the German and Austro-Hungarian railroads, about 25 000 000 were impregnated, the majority with zinc.

"As the experience of those railroads that have for from 25 to 26 years impregnated their ties with chloride of zinc under pressure, after steaming and abstracting the sap, has been very satisfactory, and as this system costs only one-half to one-third as much as impregnation with creosote or corrosive sublimate, the majority of roads have adopted the chloride of zinc process.

"The average life of ties impregnated in a rational manner with creosote or chloride of zinc under a powerful pressure reaches:

"For oak ties	19.5 years.
" fir ties	14 to 16 years.
" pine ties	8 " 10 "
" beech ties	15 " 18 "

"The results of impregnation of chloride of zinc and creosote are about equal. But, as the impregnation with creosote costs about three times as much as with chloride of zinc, a majority of the German railroads have gone over to the latter."

In addition to this favorable evidence, there was some which was unfavorable. In 1861 the Erie Railroad built a plant at Owego, N. Y., in which plank and bridge timbers, as well as a few experimental ties, were treated. The results were unsatisfactory, the large timbers in 8 or 9 years having decayed at the center, leaving a hard crust on the outside. In the opinion of the Committee, the reasons for the unfavorable results were: The original imperfections in the "Bethel process," it being attempted to extract the moisture (before injection) by the vacuum alone, without preliminary steaming; the selection of bridge timbers which are less adapted to the treatment than ties; operating on entirely unseasoned timber, and, perhaps, the use of insufficient pressure. Referring to the use of green timber:

"As a confirmation of this view, it may be mentioned that, while the bridge timber did not last well, several sets of switch ties, which had been cut about eight months and seasoned, were Burnettized at Owego for the road department of the Erie Railway, and laid in its tracks, with the result that 9 years afterward they did not show a particle of decay. These ties have been lost sight of since, but your committee has in its possession an oak tie, one of a lot of two car loads, seasoned and Burnettized at Owego, which lay in the track at Susquehanna Station for 17 years, and is almost perfectly sound to-day."

The Philadelphia, Wilmington and Baltimore Railroad and the Philadelphia and Reading Railroad built works in 1863 and 1867. The

results were satisfactory as regards decay, but the strength of the timber was so injured that the ties broke in the track.

"The explanation is that the solution was too strong, and the surplus zinc probably crystallized in the sap ducts of the timber, burst some of them asunder, and so made the wood brittle. On the Philadelphia, Wilmington and Baltimore Railroad a solution was used probably 5 or 6% strong, and on the Philadelphia and Reading Railroad the solution was $3\frac{3}{4}$% strong, while experience in Germany has thoroughly established the fact that for railroad ties the solution should not be over 2% strong, and 1.91% is considered the standard."

The present German practice, as to strength of solution, will be given later. There is no evidence known to the author to confirm the supposition that the surplus zinc contained in solutions of the strength mentioned has ever crystallized in the ducts and injured the fiber. It is probably simply a conclusion based upon an idea.

In the consideration of this record in the United States it is necessary to remember that while the use of zinc chloride as a preservative of wood was patented in 1838; the Bethel process of securing the saturation of the timber by pressure preceeded by partial vacuum was invented in 1840; and an attempt to apply the treatment was made here as early as 1850; the work for the next 20 years was largely experimental. Everything had to be learned as to the proper methods of treatment, the process perfected, and the best materials to work upon ascertained. Timber was then so cheap and plentiful in the United States as to leave but small incentive to devise means of economizing it. That the results of treatment were so largely favorable and the probable causes of failure so evident, is remarkable. The record of the hemlock and tamarack ties on the Rock Island road is difficult to account for, in the light of later experience. After 15 years' service, 75%were reported as still in the track. As will be seen later, the more recent hemlock ties on this road show a shorter average life, and it must be that the previous statement was in error, or the treatment the ties received before and after they were put into the track was more favorable than that of later years.

An endeavor has been made to learn the facts as to the method of treatment followed with these ties, but the only person accessible who had personal relations with the work in 1866 disclaims all recollection of the details and no written records were kept. The author has been told, however, that these ties were cut in December, cross-piled to

season until the following October, then treated and allowed to dry before they were placed in the track. If this is true, their record is probably fully accounted for and can be duplicated if the same conditions are supplied.

Councillor Funk's statement that the results of impregnation with creosote and with chloride of zinc were about equal, is evidently in error. In the "Organ of the Progress of Railroads," series 1897, published in Wiesbaden, there is a table in which the average duration of various ties on the German railroads is given for the zinc treatment and also with tar oil (creosoted). These are:

Oak ties, treated with zinc chloride,	15	years; with tar oil	24	years.			
Beech	"	"	9	"	"	30	"
Pine	"	"	12	"	"	20	"

Progress Since 1885.

At the time of the report to the Society, three plants treating timber with zinc chloride were in operation in the United States, each working with a different process; the difference being in the methods adopted to prevent the impairment of the preservative treatment by the washing out of the zinc salt, which is quite soluble in water. This solubility of the chloride is the only objection to its use, but the exact value of this point is not determined to the author's satisfaction. The evidence already adduced relates to the Burnettizing process proper, in which the zinc chloride alone is injected, with no attempt at any artificial method of preventing its being washed out. However, various methods have been devised to eliminate any objection on this score, and each of the plants in operation in 1885 was working on patented ideas, with this object in view.

The Thilmany Process.

The Thilmany process consisted of the injection of sulphate of zinc or copper, followed by a second injection of chloride of barium, when a chemical change was supposed to take place in the wood ducts, the chloride of barium being changed into an insoluble salt, sulphate of baryta; its chlorine forming chloride of zinc or copper. The insoluble salt was to prevent the soluble one from being washed out. Timber was treated in this way at plants in Defiance, Ohio, and Milwaukee, Wis., and some favorable results were reported. Ties had been treated

for the Chicago and Alton Railroad in 1880, and for the Erie Railroad in 1882. The Government report of Forestry (1884) states that 200 000 soft wood ties treated by this process had been put into the tracks of the Lake Shore and Michigan Southern Railroad. Some were also treated for the Wabash Railroad.

Correct records of the ties on none of these roads are obtainable. Mr. H. C. Draper, Chief Engineer of the Chicago and Alton Railroad, recollects having made an examination of the results on his road which was unfavorable to the ties. Those treated had been of various soft woods growing along the line, and the experiments were not considered successful. No definite information as to the life of these ties can be secured.

On the Erie and the Wabash railroads the results were unsatisfactory, as evidenced by letters of Mr. Charles Latimer and Mr. W. S. Lincoln, M. Am. Soc. C. E., the Chief Engineers of these roads. It is supposed that the failures were, in part, due to the expected chemical action not taking place when the substances were confined to such small spaces as the ducts of timber.

The Zinc-Gypsum Process.

The zinc-gypsum process consisted of one injection of a solution of chloride of zinc and gypsum. It was claimed that the gypsum crystallized in the ducts of the wood, thus stopping them up and preventing the zinc from being washed out. As the process was then new, no evidence was submitted in the report, except statements of chemists showing the apparent impossibility of washing the zinc out in laboratory tests. A plant was operated at St. Louis for several years, but was not a success financially; it was dismantled, and efforts to secure any facts as to the value of the work done have been unsuccessful.

The Wellhouse or Zinc-Tannin Process.

The Wellhouse or zinc-tannin process consisted of the injection of a solution of zinc chloride containing a small amount of glue, followed by another injection of a solution of tannin, the effect of the tannin, it was claimed, being to form with the glue small particles of artificial leather, insoluble in water, which would plug up the ducts and retain the zinc chloride. A plant was in operation in St. Louis under the management of Mr. J. P. Card. The first timber treated was in

TABLE No. 1.—Examination of Ties on the Atchison, Topeka and Santa Fé Railroad.

At Topeka.	July 2d, 1891.				
Kind of wood.	No. of ties.	Date laid.	No. now in track.	Percentage.	Years in track.
Colorado pine	49	July 7th, 1881	Destroyed by wreck.		
Cottonwood	87	August 1st, 1882	43	49.4	8.9
Sweet gum	51	" "	10	19.6	8.9
Red oak	48	August 18th, 1882	29	60.4	8.8

At La Junta.			July 6th, 1891.			May 11th, 1893.			March 14th, 1897.		
Kind of wood.	No. of ties.	Date laid.	No. now in track.	Percentage.	Years laid.	No. now in track.	Percentage.	Years laid.	No. now in track.	Percentage.	Years laid.
Black oak	49	May, 1882	49	100	9.1	47	96	11	44	90	14.8
White cottonwood	49	" "	47	96	9.1	11	22.4	11	8	16	14.8
Cottonwood	50	" "	46	92	9.1	22	44	11	17	34	14.8
Colorado pine	50	July, 1881	50	100	10.0	20	40	11.8	None.	.....	.....

1879. In the following years paving blocks, experimental lots of ties for various railroads and miscellaneous timber were treated. The ties were laid in the vicinity of St. Louis and on the Atchison, Topeka and Santa Fé Railroad. Regarding this work O. Chanute, M. Am. Soc. C. E., says:

"In 1885 I went to St. Louis upon the occasion of the repairing of the St. Louis Bridge. This had been laid in 1879 with gum-wood blocks prepared by the Wellhouse process. These blocks, originally 5 inches thick, had worn down so thin as to give trouble, but they were sound after 6 years' exposure. Less than a dozen were found decayed in the whole length of the bridge. The Burnettized pine stringers were also sound, and the local railroads reported ties treated by the Wellhouse process to be a success, except one lot laid in mine screenings and 'bone' at East St. Louis."

Of the experimental ties put into the tracks of the Atchison, Topeka and Santa Fé Railroad in 1881 and 1882 examinations were made by Mr. Chanute in July, 1891; of those at La Junta, Colo., and Topeka, Kans., and in May, 1893, and March, 1897, at La Junta. James Dun, M. Am. Soc. C. E., kindly had examinations made in both places in December, 1898. He found some of the treated ties still in

the track, but the tin tags identifying each tie had entirely disappeared, or had become so badly scarred that no count of any value could be made. The records of the previous examinations are given in Table No. 1 (page 119).

Atchison, Topeka and Santa Fé Railroad Works.

The first road to adopt the use of the treated ties, other than as an experiment, was the Atchison, Topeka and Santa Fé Railroad, of which A. A. Robinson, M. Am. Soc. C. E., was then Chief Engineer. This road, in 1885, had a plant built, under Mr. J. P. Card's direction, at Las Vegas, N. Mex., consisting of two retorts 6 ft. in diameter and 106 ft. long, which operated on mountain pine. The Wellhouse or zinc-tannin process was used except from 1889 to 1893, when the Burnettizing (proper) method was followed. The number of ties treated was as follows:

1885	111 503	1892	187 270
1886	333 298	1893	178 167
1887	189 386	1894	247 566
1888	285 104	1895	256 887
1889	169 174	1896	336 070
1890	105 753	1897	380 523
1891	128 568		
Total	2 909 269		

These ties have been used from Dodge City, Kans., to El Paso, Tex., and Silver City, N. Mex.; and a few in Eastern Kansas and Oklahoma Territory. The majority, however, have been used in New Mexico. The life of the untreated mountain pine is given as not to exceed 4 years. Complete records of the treated ties are unobtainable, but through the kindness of Mr. James Dun, Chief Engineer; Mr. H. U. Mudge, General Superintendent, and Mr. George W. Noyes, Manager of the Las Vegas works, the following facts have been secured:

"The New Mexico Division was built in 1880, with native untreated ties. We commenced renewals in 1885, with native pine treated ties, and since that time no other class of ties has been used. The untreated ties were, of course, all removed prior to 1893, so that ties taken out since that time are evidently all treated ties.

"We have assumed that after 100% of the original ties have been removed, all ties taken out were then treated ties. For the 5 years, 1893 to 1897, inclusive, we inserted on the New Mexico Division, on 380 miles, 414 189 treated ties, an average of about 217 ties per mile per year. The renewals for this period were about normal, the ties in this track being in very good condition, therefore, the average life of these ties can be put down as 11 or 12 years.

"As to peculiarities in the length of life of ties due to the character of the soil, I have to say that in New Mexico, ties last much longer in rock ballast than in soil, and decay very much faster in sandy soil than in clay. There are still (July, 1898) a great many treated ties in track branded 1885. You will also understand that a good many of the ties removed are taken from sharp curves and were removed on account of being spike killed.

"On the New Mexico Division we are using treated native pine ties. On other divisions we have been using cedar and oak; usually oak in main lines only. The life of oak ties in heavy main line service is about 6 years; of cedar ties in light branch service, about 10 years.

"Relative to the Burnettized ties laid in 1889 to 1893, we have as yet been unable to demonstrate whether the life of these ties is less than that of the Wellhouse process, as ties laid in 1889 have not yet shown signs of deterioration."

Beginning in 1897, the records of the ties on the New Mexico Division have been more carefully kept, and for future years they will show the number of removals and the causes. Mr. J. E. Hurley, Division Superintendent, has furnished a statement (Table No. 2) covering the years 1897 and 1898.

TABLE No. 2.—Treated Ties Removed on the New Mexico Division of the Atchison, Topeka and Santa Fé Railroad in the Last 9 Months of 1897 and in 1898.

Year removed.	Years of Treatment.												Total removed.	Average life.
	1885.	1886.	1887.	1888.	1889.	1890.	1891.	1892.	1893.	1894.	1895.	1896.		
1897...	13 114	17 540	15 527	10 327	2 459	2 552	566	560	55	104	17	10	62 831	10¼ yrs.
1898...	14 580	17 230	24 662	18 424	5 575	3 649	1 917	5 444	1 900	274	118	49	93 822	10½ "
	27 694	34 770	40 189	28 751	8 034	6 201	2 483	6 004	1 955	378	135	59	.....	.....

"I am not able to give you the exact number removed from different causes, but would say that 1% of the total will cover all ties removed on account of being broken by wrecks and causes of a like

nature, the balance being removed on account of being rotten, worn out," etc.

The average life of the ties removed does not quite equal that estimated for them by Mr. Mudge, but it does not follow that his estimate is in error; although his premises apparently are. With the renewal of 62 831 treated ties in 9 months of 1897, and of 93 822 in 1898 it is improbable that the 414 189 ties renewed in the 5 years, 1893 to 1897 inclusive, could all have been treated ties. It will be noted that the average life of the ties removed in 1898 was longer than that of those removed during 1897; and it is understood that the 1885 ties are not yet all removed. It is natural to expect that the life of the ties removed each year will continue to increase for some years to come, certainly until all the 1885 ties are removed. As the best results can hardly be expected from the treatment of the first year, it is very probable that 12 years' life will be reached by these ties.

In the Las Vegas works, ties are steamed from 2 to 6 hours, depending upon their condition when received, green timber requiring longer time. They are in a vacuum for ½ to 1 hour, and under pressure for 2½ to 3 hours. Prior to 1890 the chloride was prepared at the works, but since then it has been purchased, in a fused condition. The purity has varied from 95 to 97 per cent. "The amount of zinc chloride injected varies in different years, varying from 0.28 to 0.47 lbs. of chloride per cubic foot of timber treated, which includes sawn lumber of all kinds and piling." Seasoned timber is preferred; and after treatment ties are at once shipped to their destination. They are then supposed to be unloaded, piled up in lots of ten, in triangular piles, so arranged that only one end of any tie is on the ground; and allowed to season for about 30 days. In point of fact, they are generally put into the ground immediately, full of water; on account of the difficulty in providing for the demands of the road with the present facilities. In Mr. Noyes' opinion, to secure the best results, ties should be seasoned for 6 months before treatment and for the same time thereafter.

In 1896 this plant was enlarged by the addition of a third retort. In 1897 a large plant, consisting of four retorts, was constructed by private parties, at Somerville, Tex., where it is expected to treat large quantities of ties for the Santa Fé system; and this railroad has just completed another plant at Bellemont, Ariz., consisting of two

retorts, where ties will be treated for that part of the system. The officials of the road express themselves as satisfied with the value of the treatment, but are discussing the relative values of the zinc-tannin and the plain zinc-chloride methods. The additional cost to inject the glue and tannin amounts to several cents per tie; and they are in doubt from their own experience whether the increased cost results in an increased life, and, if so, to what extent.

Union Pacific Railroad Works.

The Union Pacific Railroad had some experience with wood preservation in 1867 and 1868 when works were erected at Omaha and operated on cottonwood ties for 1 year, when they were discontinued as it was impracticable with the plant at hand to prepare more than one-tenth of the ties required.

In 1886 new works, consisting of two retorts, were erected at Laramie, Wyo., under Mr. Chanute's direction. They were operated for 2 years by the railroad, and 242 000 ties were treated, when work was suspended for reasons of temporary economy. It was also reported to have been because the United States Government stopped the cutting of ties on its lands, and because of prejudice on the part of some one in authority. Whatever the reason, the works were shut up, were partially burned in 1889, and were then dismantled.

While operated, the Wellhouse process was used. According to the detailed record of the operation of this plant from July 26th to August 21st, 1886, the strength of the solution of chloride corresponded to 1.90 on the Beaumé scale, representing 1.46% of chloride in the solution, and the absorption of chloride was 0.65% of the weight of the tie, or 0.78 lb. per tie, the timber (mountain pine) weighing 29.51 lbs. per cubic foot.

Mr. Wm. Ashton, Division Engineer of the road, in a letter to Mr. Chanute, dated January 31st, 1898, speaking of the preserved ties, says :

" This work was done prior to my assignment to this portion of the line, consequently I have not had the opportunity to watch the service given by these ties fully as I would have liked. For the past 3 or 4 years I have been observing what remained of them and have talked frequently with our roadmasters, one of whom I understand has been on the portion of the line on which these ties were laid since they were placed in the track. From what I know and have been able to gather concerning these ties would state as follows:

"Ties treated were native pine and spruce, Oregon fir, elm, ash, maple and oak, by far the larger number being of native pine and spruce.

"The native pine and spruce ties began to fail in 1893; those removed during this year seemed to be mostly 'top-cut' ties. In 1894 most of the top-cut and middle-cut ties were taken out, while in 1895 and 1896 practically all the pine and spruce ties in the main track were removed. These ties were not cut by the base of the rail as badly as untreated ties of the same timber, and their failure could seldom be attributed to this cause. It was observed that the heart and bottom of these ties failed first, leaving a hard shell-like top. There are still a number of these ties in sidings.

"The Oregon fir ties were usually small, 7 x 8 ins., and were found to split badly in spiking. They failed a year or two sooner than the native pine and spruce ties.

"The elm, ash and maple ties have all been taken out during the past 4 years. The service of these ties would have seemed to be about the same as those of native pine and spruce.

"A few of the oak ties are still in the main track. We find they are getting very brittle and are breaking badly. They will probably all be taken out during the years 1898 and 1899."

The above report indicates an average life, for the mountain pines and spruces in the main track, of about 9 years. In January, 1896, Mr. Ashton read a paper before the Denver Society of Civil Engineers on the "Average Life of Cross-Ties on Steam Railroads in Colorado."*

In this paper he says :

"Under ordinary circumstances my experience on the lines in Colorado has been that ties of Colorado pine, balsam and white spruce will last in track 3½ to 4 years under ordinarily heavy traffic. Under light traffic on a well-tied track, they will probably average 4 years. On our main line we usually take out about half of these ties after 3 years' service, and the balance the fourth year."

This would indicate the result of treatment on these ties to be an unqualified success, although the life attained is materially shorter than that reported for the Atchison, Topeka and Santa Fé Railroad, with practically the same material, but perhaps a drier exposure.

At the author's request, Mr. Berry, the present Chief Engineer, had an analysis made of the chloride of zinc remaining in one of the mountain pine ties taken out of the track in 1898, after 11 or 12 years' service. The analysis showed 0.146% by weight of the chloride, the amount being the same in samples from the center and from near

* *Journal* of the Association of Engineering Societies, Sept., 1897.

the outside of the tie. In the report of Mr. W. L. Robinson, chemist of the road, he says: "The ties, when first taken from the works, seldom carried over 0.25% of metallic zinc" (or 0.52% of zinc chloride). "The ties (analyzed) are in a very good state of preservation."

Works at Chicago, Ill.

This plant is the successor of the St. Louis plant, in which the Wellhouse process was originally worked by Mr. J. P. Card, and was built in 1886 by Mr. Card and Mr. Chanute, operating under the name of The Chicago Tie Preserving Company. It consisted first of two retorts, to which a third was added in 1891 and a fourth in 1894, and has treated, up to December 31st, 1898, 4 007 946 ties for the Chicago, Rock Island and Pacific Railroad, besides small lots for other parties. The usual methods of the Wellhouse process were followed until 1896, when a modification was introduced, consisting of the omission of the glue from the zinc chloride solution and the subsequent injection of the glue separately, followed by a third injection of the tannin.

This change was based upon experiments, and the idea that the admixture of glue with the chloride solution lessened its fluidity and consequently increased the difficulty of securing the injection of the desired amount of the chloride. It might also be argued that the province of the glue being with the tannin simply to plug up the ducts to prevent the washing out of the zinc, the only proper place for it is at the ends of the ducts and medullary rays; and that it is unwise to introduce into the body of a tie perishable organic material, like glue, to increase the duty of the chloride, and where it is practically impossible to force the tannin to it, by a subsequent injection. This modification, by necessitating three injections in place of two, somewhat increases the expense of the treatment, and while it is undoubtedly an improvement, the period of its use has been too short to demonstrate it, in increased life. The best evidence of its value is in the increased absorption of chloride obtainable.

The ties treated for the Rock Island road have been entirely hemlock and tamarack. The number by years is given in Table No. 4. The number of tamaracks being relatively small, no distinction between them is made in the following, except where the tamaracks are specifically referred to.

It is extremely unfortunate that the records of service of these ties are incomplete and of doubtful value. It is stated that practically none of the ties was removed from the track until 1892. At the time they began to come out, instructions were issued by the management of the road, that all treated ties removed should be piled up and not destroyed until inspected by a representative of the Tie Preserving Company. This was done in 1892 and 1893, except for those removed from a lot of 39 604 shipped west of the Missouri River in 1886; the removals in 1892 being 2 319 or 2.1% of the ties laid 6 years before east of the Missouri River; removals in 1893, 14 282, or 3.8% of those laid 6 and 7 years before, in 1886 and 1887, or 13% of those laid in 1886. The removals for these 2 years, 16 601 ties, represent 15.3% of those laid in 1886 east of the Missouri River, so that at this time there were still in service 84.7% of the ties laid 7 years before.

In the years subsequent to 1893, the ties removed were not inspected and counted by an agent of the Chicago Tie Preserving Company, but were reported by the section foremen to the road-masters, and by the latter to the Superintendent of Maintenance of Way, who compiled a general statement. The removals, as given for the tracks east of the Missouri River for the years 1894 to 1897, are shown in Table No. 4. From these statements of the annual removals it was computed by the Tie Preserving Company that the average life of the treated ties was $10\frac{1}{4}$ years, an increasing percentage coming out each year, up to the twelfth, when all were supposed to have been removed.

Within the past year doubt has been thrown by some of the railroad officials, upon the accuracy of these yearly returns, and it has been claimed that the ties were averaging less than 7 years. Unfortunately, the ties were not marked in any way to indicate the year treated, until quite recently, and there had been very considerable changes in the divisions of the road, in the sections and in the methods of managing the track system. The records have become inextricably mixed, so that it is impossible to determine when and where the earlier treated ties were laid. This reduced the question of the average life to a matter of recollection and opinion, and while opinion had been favorable up to 1894, it now swung the other way. Finally, in July, 1898, Mr. H. A. Parker, Second Vice-President and Chief Engineer, ordered a count by sections to be made on August 1st, to determine how many

ties of each kind (oak, cedar and treated hemlock) were then in the tracks of the road, each section foreman counting the ties on his own section.

The result of this count is shown in Table No. 3.

TABLE No. 3.—Ties in Tracks of Chicago, Rock Island and Pacific Railroad, August 1st, 1898.

Kind of tie.	White oak.	Cedar.	Treated hemlock.	Pine.	Treated Pine.	Totals.
In main track.....	4 541 235	3 308 675	2 545 240	29 249	93 940	10 518 339
" side "	484 626	877 143	345 948	4 586	10 010	1 722 313
Totals.....	5 025 861	4 185 818	2 891 188	33 835	103 950	12 240 652
Percentage in sidings.....	9.7	21	12	13.5	9.7	14.1

The totals in Table No. 3 represent averages of 2 430 ties per mile in sidings, 3 050 ties per mile in main track and 2 950 ties per mile in all tracks. Mr. Parker is satisfied that this total number of all ties is substantially correct. The treated pine ties shown in the table were treated at works in Texas.

It remains to deduce from these figures the probable life of the treated ties. Up to the same date (August 1st, 1898) there had been treated, hemlock and tamarack ties to the number of	3 860 295
Less, shipped to the Rock Island and Peoria Railroad and not in above count.....	72 194
Total to account for.....	3 788 101
" counted in track.....	2 891 188
Balance presumably renewed	896 913
The treated ties were at that time being shipped west of the Missouri River, a distance of from 500 to 1 000 miles, and many were in transit, there having been shipped from the treating works during the last ten days of July, 29 209 ties; and there were also, certainly, some distributed but not in track. Assuming as fair to deduct for these two items.....	50 000
Leaves, as the probable renewals	846 913

This absorbs all the ties treated up to 9 years previous, or to the close of the treating season of 1889, amounting to 825 796, of which 21 623 had been shipped to the Rock Island and Peoria Railroad, so that the tracks under discussion received 804 173.

Leaving, presumably, to come from the ties treated in the following year, 1890, 42 740.

This number is 17% of the ties treated in 1890 and is to be deducted from the 9 years, leaving 8.83 years as the probable average life of these treated ties, provided there was no material error in the general count made in August.

The author does not consider this count conclusive as to the life of the treated ties on this road. The time thus determined is shorter than indicated by the removals during 1892 and 1893, in the latter year there being still in the track 85% of the ties laid 7 years before; and as indicated by ties laid on four sections of the same road west of the Missouri River; as well as by those tested on the Pennsylvania Railroad and the Duluth and Iron Range Railroad, as described later. It is extremely difficult to distinguish the treated hemlock and tamarack from cedar, by surface inspection in the track, if indeed it is possible to do so in many cases without cutting the tie, and it seems quite possible that many treated ties may have been reported as cedar; but in the author's opinion based on personal experience with the ties, the mistaking of cedars for hemlocks and tamaracks would not be as likely to occur. The only reason given for doubting the annual reports was that they were based on the reports of the section foremen, and had been made in the regular course of work, without any especial effort to secure accuracy. Since the August count, however, orders have been issued to keep these records accurately, and this paper has been held open until the reports of the renewals for the year 1898 became accessible, in order that the probable value of the previous year's reports could be gauged by this one. Table No. 4 gives this as well as all other data that can be secured. It will be noted that the total renewals on the lines east of the Missouri River are 481 947. The total number of treated ties put into the same tracks for the years 1886, 1887 and half of 1888 are 461 765; so that, if the annual reports of ties removed are correct, all ties put in since about the middle of 1888 are still represented in service, and the average life of over 10 years, as figured by the Chicago Tie Preserving Company,

is correct. Since the above was written it has been discovered that the treated tamaracks in the tracks west of the river have been confounded with the treated pines and reported as such—to what extent is unknown. While the number so omitted from the count of the "hemlocks" cannot be very large, it may be 50 000, and the life of the treated ties would be correspondingly increased.

All these figures, based on removals without any knowledge of the year of treatment of the particular ties removed, are extremely unsatisfactory and unscientific; but they are the best obtainable and are so presented.

TABLE NO. 4.—TREATED TIE RECORD, CHICAGO, ROCK ISLAND AND PACIFIC RAILROAD.

Year.	No. treated.	Shipped west of Missouri River.	Shipped to R. I. & P. R. R.	Remainder charged to track east of Missouri River.	Removals from tracks east of Missouri River.
1886.....	148 816	39 604	8 048	101 164	.....
1887.....	265 377	.....	3 060	262 317	.....
1888.....	201 272	.....	4 703	196 569	.....
1889.....	210 331	.....	5 812	204 519	.....
1890.....	253 245	.....	27 271	225 974	.....
1891.....	339 423	981	23 300	315 142	.....
1892.....	325 736	15 123	.....	310 613	2 319
1893.....	405 267	100 000 (?)	.....	305 267	14 282
1894.....	430 362	110 048	.....	320 314	33 344
1895.....	368 630	225 669	.....	142 961	62 245
1896.....	297 252	213 613	.....	83 639	71 695
1897.....	425 459	324 876	.....	100 583	137 708
1898.....	336 776	283 612	.....	53 164	160 354
Total.....	4 007 946	1 313 526	72 194	2 622 226	481 947

In 1886 a large number of treated hemlocks and untreated white oaks was laid in the tracks on Sections 1, 2, 5 and 9, west of the Missouri River. The renewals on these were reported in the years 1891, 1892 and 1893.

Inquiries in 1898 showed that no later separate accounts had been kept, but the Roadmaster, Mr. J. D. Sullivan, in September of that year, had a careful count made to determine the number then in the track. These various figures are given in Table No. 5. It will be noted that on Sections 1 and 2, 28.9%, and on Sections 5 and 9, 58.8%, of the treated hemlocks were still in service after 12 years. Mr. Sullivan adds "the ties still in the track are in fair state of preservation, and the best will last 4 years more." The record of the oaks is also given in Table No. 5.

TABLE No. 5.

Section.	No. of ties.	Renewed in 1891.		Renewed in 1892.		Renewed in 1893.		No. in Track Sept., 1898.		Percentage removed.	
		No.	Percentage.	No.	Percentage.	No.	Percentage.	No.	Percentage.		
					Treated Hemlock.						
1....	8 000	0	0	0	0	200	2.5	1 488	18.6	81.4	Average, 71.1
2....	8 000	0	0	0	0	425	5.3	3 137	39.2	60.8	
5....	3 050	0	0	0	0	67	2.2	1 963	64.4	35.6	Average, 41.2
9....	2 800	0	0	0	0	0	0	1 477	52.7	47.2	
	21 850	0	0	0	0	692	3.2	8 065	36.9	63.1	
					Natural White Oak.						
1....	10 000	0	0	487	4.9	800	8.0	3 184	31.8	68.2	Average, 74.1
2....	10 000	418	4.2	658	6.6	1 120	11.2	1 983	19.8	80.2	
5....	15 000	150	1.0	576	3.8	805	5.4	2 732	18.2	81.8	Average, 74.5
9....	15 200	282	1.8	685	4.5	1 335	8.8	4 979	32.7	67.3	
	50 200	850	1.7	2 406	4.8	4 060	8.1	12 878	25.6	74.4	

Mr. Sullivan states that in 2 years practically all the oaks will have been removed.

On Sections 1 and 2, 71% of the hemlocks and 74% of the oaks had been removed after 12 years and on Sections 5 and 9, 41% of the hemlocks and 74% of the oaks. As to the variation in the life of the hemlock on the different sections, Mr. Sullivan says:

"The cause is the difference in dirt and ballast, Sections 1 and 2 being close to the Missouri River, and dirt, largely silt and sand, with 3 miles of rock ballast in the former. Sections 5 and 9 are 24 and 48 miles west of the river, and the ground is largely black firm prairie loam. All kinds of woods, except cedar, are short-lived in river bottom and bluffs."

If the hemlocks now in the tracks last 4 years longer, for the best, as Mr. Sullivan states, it is fair to put the average remaining life of all the ties now in at 2 years. On this basis the average life of all the treated ties would figure for

Section 1..... 10.27 years.
" 2..... 11.13 "
" 5..... 12.34 "
" 9..... 11.62 "

or for all four sections 11 years. These averages are derived thus, taking that for the four sections as an example:

Renewed in 1893, after 7 years' service	3.16	×	7	= 22.12
" 1894, 1895, 1896, 1897 and 1898 (average 9½ years)	59.94	×	9½	= 569.43
Remaining in track now, good for 2 years more	36.9	×	14	= 516.60
	100.00	×	11.08	= 1108.15

Average, 11.08 years.

The life of the treated ties on this road has undoubtedly been less than is readily obtainable by a more rational practice than was the custom prior to 1895. Until then the Tie Preserving Company treated all ties sent to it, regardless of their condition as to moisture or soundness. Since 1895 this has been changed, and the ties are re-inspected, when received at the works, by the Preserving Company's men, and if not sound are condemned and turned back to the tie contractor; and if the ties are too wet for treatment, they are piled up until sufficiently dry to take the proper amount of chloride.

In 1890 the Chicago Tie Preserving Company treated some experimental ties for the Duluth and Iron Range Railroad. These consisted of 85 white pines, 85 tamaracks, and 86 Norway pines. They were cut during the winter of 1889-90, treated in October, 1890, and placed in track very soon after, together with 10 Norway pine, 10 white pine and 10 tamarack untreated ties, which were cut at the same time and off the same land as those subjected to treatment.

"The roadbed in which these ties were laid is in glacial drift and is what you might call gravelly hard-pan, carrying some lime, and the drainage is good; the ballast is a fairly clean and reasonably coarse gravel."

Mr. J. L. Greatsinger, President and General Manager, had an examination of these ties made in February, 1898, and wrote Mr. Chanute as follows:

"In the winter of 1895 and 1896, we were obliged to tear up the track with dynamite where these ties were originally laid. In doing so, very few of them were broken, but nearly all of them went through the ordeal remarkably well and were relaid in double track in the spring of 1896, as the grade at this point was lowered about 8 ft. At this time an inspection was made of them, and it was decided to lay part of them the same side up as before, while a few of each kind were reversed; and as the same rails, 80 lbs., were used in relaying, it did

not necessitate, except in a few instances, making new spike holes, and where this was done the old holes were plugged.

"While we have but 5 or 6 ins. of snow, yet it is difficult to give as close a report on their condition at this time as it will be next spring, but I am now able to report as follows:

"*White Pine (One Notch).*—The rail has cut into the tie but very little, the greatest depth being $\frac{1}{8}$ in. By cutting into the top of one of the ties, the wood appeared apparently as good as new.

"*Tamarack (Two Notches).*—The rail has cut into them a little more than the white pine; about $\frac{1}{4}$ in. All sound, and otherwise appear as good as new.

"*Norway (Three Notches).*—The rail has cut into these about the same as tamarack, a $\frac{1}{4}$-in. cut in the top of the tie. Found the wood in fair condition, but not as good as either tamarack or white pine.

"In general terms, the process of preservation has not only prevented decay, but has made the wood much denser, thus materially reducing the cutting usually done by the base of rail, and judging from the results already attained, these ties are yet good for 15 years, and, had we used tie plates, for a much longer time, as the destruction of the tie will be from wear and not from decay.

"Next spring I will take out some of each kind and send you some sections of them under the rail bearing and through the spike holes, which will enable you to see the effect.

"I send you by express, some pieces that were cut from the ties the first of this month, for your inspection. I consider this a good showing, as our service on ties is the most severe of any I know of in this country, as there have been times when we have averaged for every day in the month over 100 trains passing over them every 24 hours. Our heaviest standard locomotive, ready for service, weighs about 120 tons, and our average ore car, loaded, carrying 24 gross tons in a car, itself weighing about 24 000 lbs.; length over all, 24 feet."

In response to an inquiry by the author, Mr. Robert Angst, Chief Engineer, under date of June 22d, 1898, writes:

"The untreated ties you inquire about have reached the end of their career, and are now in the ash heap. They were a fair average of the untreated ties on our line, the effective life being between 7 and 8 years."

In 1891, 200 tamarack and 200 hemlock ties were treated for, and placed in, the tracks of the Pittsburg, Fort Wayne and Chicago Railway. Thomas Rodd, M. Am. Soc. C. E., Chief Engineer, says of them (1898):

"After these ties were put in the track we watched them pretty carefully and for about 3 years they cut rather more than an oak tie.

After that, however, they seemed to cut less than an oak tie, and are to-day in good shape in our main track."

In the spring of 1898 two of these ties had decayed, and were removed. An examination after removal satisfied Mr. Chanute that the two ties were from dead trees, and their failure after 7 years' service he attributes thereto. In 1896 and 1897 the same road put in track about 50 000 hemlock preserved ties; and during 1898 there have been treated for it 13 000 beech and 11 000 black and red oaks.

The Maintenance of Way Committee of the Association of Transportation Officers of the Pennsylvania Lines made a full report on the subject of ties 2 or 3 years ago, in which it was recommended that 100 000 ties, preserved with zinc chloride, should be laid each year on their lines.

There have been a number of ties treated for the Rock Island and Peoria Railroad, a subsidiary line of the Chicago, Rock Island and Pacific Railroad, operated under separate management. An effort was made to secure data from this road as to its experience with these ties, but nothing but very general statements could be had. From the Chicago, Rock Island and Pacific Railroad it was learned that the number of treated ties delivered to the road was as follows:

1886	8 048
1887	3 060
1888	4 703
1889	5 812
1890	27 271
1891	23 300
Total	72 194

In response to an inquiry, Mr. J. E. Breckenridge, Roadmaster, wrote, under date of June 27th, 1898:

"Our ballast at that time" (when the ties were put in) "was mostly gravel, and cinders in places on top of gravel. On prairie soil, gravel ballast, ties lasted about 4 years; on timber soil, gravel ballast, about 7 years; around coal shafts with slack ballast, about 3 years. I think those ties might have done better had they been piled up for one year and got thoroughly dry. The ties seemed to come to us from the vat, and were generally put in track very soon after they were received."

In reply to a later inquiry as to the number and location of any ties still in track, Mr. Breckenridge wrote: "There are none of the ties you speak of in track now."

The author could see no reason to expect any such short life for the ties as this indicated, judging from the experience of others with them; and as inquiries of the higher officials of the road produced similar views to those of Mr. Breckenridge, it was determined to test by actual examination and count, the statement as to the ties being all out; and to determine the reason, if such was the fact. Accordingly, S. McM. Rowe, M. Am. Soc. C. E., who was for some time in charge of the Las Vegas treating works, was engaged to walk over the entire line of 118 miles and count all the treated ties he could find. Mr. Rowe made this examination in November and December, 1898, and in his report practically gave the number of ties on each mile separately, so that his count could be readily verified or disproved. He absolutely identified 20 272 treated ties and estimated that there were 2 000 more covered by snow or surplus ballast—practically 30% still in service—indicating how easy it is to lose the reckoning of ties in track. Mr. Rowe states:

"The ballast consists about as follows: From Rock Island yard to Milan, mostly sand, quite fine, taken from local sand pit. The main line eastward as far as Toulon is mainly fine gravel, some places very profuse, covering the ties many places so as to prevent inspection, while much of the distance the ballast is thin.

"Almost all of the coal branch, and that portion of the main line east of Toulon, is ballasted with burnt clay, a very good material for the purpose. The burnt clay ballast is in ample quantities, covering the track entirely for considerable distances and banked against the ends of the ties so as to materially retard drainage.

"In consequence of this, the track is down and ties are churning in the softened track. To what extent this has affected the lasting qualities of the cross-ties, I am not prepared to say, but the matter is here noticed with reference to that question.

"The identification of the treated hemlock ties was not difficult, as the grain of the wood, the presence of bits of the bark and the well-defined shade of the surface of the timber, were all an aid in this; but the most striking characteristic of the hemlock was what I would call its 'integrity'; that appearance of soundness of wood, and especially the absence of material wear under the rail. To fully understand this, a personal examination is necessary, as while the superiority of these treated ties is so marked, the actual condition is hard to put into

words to express it fully. This is especially noticeable between Crampton and Orion on the Cable Branch.

"In regard to the identification of the tamarack it was found very difficult, as the difference of the grain from that of the cedar was less marked, and only the greater firmness of the wood and the shade of color due to the treatment were available, and, in consequence, many were missed in checking.

"Without a more searching examination of these ties yet in track, it is difficult to form an opinion as to the time they will serve. I would judge that perhaps 25% would be removed in a relaying of the track, and that the service of the other 75% would be from 2 to 5 years if properly handled. Very few of the ties (hemlock) were found in the waste piles.

"The resistance to rail wear was remarkable, and when the hemlock was laid with oak, the wear was inappreciable. When laid here and there, interspersed with cedar, the wear was greater, but still insignificant compared to that of the cedar, and this fact aided very much in finding them."

Southern Pacific Railroad.

The profession and the railroads owe a great debt to Mr. J. Kruttschnitt, General Manager of the Southern Pacific Company, for the very careful and complete manner in which he has had kept the records of the ties treated and used on his road. His is the only company which can furnish complete data on the subject; and when the labor entailed is considered, especially after preliminary tests have indicated the success of the treatment, so that there is some confidence in it, it is the more remarkable that he has so consistently and constantly devoted so much of his own time to it, as is necessary to insure its continuance. There can be no question of the value of these records to the railroad, and the slight expense is thoroughly justified, and would be, even if much greater than it is. One of the difficulties in connection with the treatment of ties in the United States in the near future is likely to be this one of proper records. Treated ties, like the untreated ones, will show great, and, to the casual or superficial observer, entirely unaccountable, variations in life; and, unless it is possible to determine definitely by total averages the life secured from the ties, there are apt to be periods when those in authority will conclude—from impressions, not records—that treatment is a failure and a waste of money. Even under favorable conditions, ties will develop eccentricities which will demand investigation

and expert consideration, if the best results are to be obtained. This has been the case already in several instances in the United States and abroad, and as more roads adopt the process and attempt to let it run itself, failures are bound to occur, without the records which, if properly kept, would indicate the cause and permit the remedy to be applied.

All the treated ties on the Southern Pacific Railroad, except the bridge ties, which are creosoted, are preserved by the old original Burnettizing method, viz., the injection of chloride of zinc only, with no other treatment to prevent the leaching out thereof. The use of these ties was commenced on the Texas and New Orleans road, a part of the Southern Pacific, Atlantic System, in 1887, and until 1891 they were treated at the works of the Houston and Texas Central Railroad, at Houston, Texas; after that date, at the plant of the Southern Pacific road, erected near the same place.

The operations on this road are described in a very complete and thoughtful report made to Mr. Kruttschnitt by Mr. J. T. Mahl, Engineer of Maintenance of Way of the Atlantic system, under date of July 9th, 1898. The following data are from this report, supplemented by information in letters by Mr. Kruttschnitt; the quotations are from from the report:

"The timber treated is long-leaf and short-leaf yellow pine from Eastern Texas and Western Louisiana." It is all sap wood, and, untreated, has a life of only 1½ to 3 years, being the cheapest wood obtainable. The heart wood of the same timber, untreated, would have a life of from 6 to 9 years.

"It is felled at all the seasons of the year, and no attempt is made to season it, either before cutting into ties or after receiving it at the works, previous to treatment." However, "in treating the large number of ties we are called on to treat for this company and its affiliated lines, some timber necessarily must be stored at the works, and this receives natural seasoning to some extent. When ties are treated some care is taken in charging the cylinders to fill them as nearly as possible with ties of the same age, so as to secure uniform treatment; for if green ties and ties which had been seasoned for 3 to 6 months were subjected to the same treatment, the percentage of absorption of the seasoned ties would be much greater than the absorption of the green ties."

The strength of the solution of chloride formerly used was 2½° of the Beaumé scale, corresponding to 1.7% of chloride; and from 0.70 to

0.75% by weight of the pure chloride was injected. As many ties shattered and split under traffic, it was concluded that the strength and quantity of the injection was too great, and this has been gradually reduced until the strength is now represented by $2\frac{1}{3}°$, and the quantity is from 0.50 to 0.55 per cent. In the author's opinion, this is probably a mistake. In Table No. 9 it will be noted that of 92 206 ties removed from six districts during 2 years and 4 months, 87% were rotten, and only 11% were removed because of shattering. Of course the limiting line between decay induced by shattering under the rail and that acting on the tie generally is difficult to define, and these figures may not fairly represent the relative difficulties from the two causes. This trouble with shattering has been found elsewhere with different material and with very much higher strengths and larger quantities of the chloride, without, however, developing into any very serious matter. The German practice at some works is to use a solution 3° strong, and the specifications for treatment at the J. Ruetgers Works require, in case of green or close-grained wood, into which it is impossible to inject the required amount of solution of 3° strength, that this strength shall be increased even up to $4\frac{1}{2}°$, so as to secure the presence of an equivalent amount of the zinc chloride in all ties. The Chicago Works use at times a 3.8° solution and now inject 1% of chloride; and yet, out of 4 000 000 ties treated, not one broken tie has been reported. The proportion of those shattered to pieces is unknown, but is believed to be small.

The best German expert advice is to the effect that the change in strength from $2\frac{1}{2}$ to $2\frac{1}{3}°$ is entirely too small to affect at all the strength of timber. If the shattering at any time exceeded or nearly approached the destruction by rot, it was more probably due to the use of excessive heat or impure chloride.

"With few exceptions the treated ties have been used exclusively in the main line between Houston and El Paso, a distance of about 836 miles." Tables Nos. 6, 7 and 8, give very complete data as to the life of ties on the Atlantic system, which includes the above.

To obtain the information regarding treated ties "it has been the practice of the company to stamp the ends of the ties, immediately after treatment, with a brass die giving the month and year of treatment. Track foremen have been required to put the stamped ends of all ties upon the same side of the track, that is to say, on the right hand going from New Orleans to El Paso. Foremen are provided with

an especially prepared blank on which they are required to report at the end of each month, the number of treated ties of the same date, removed by them; also the causes of removals of these ties, that is to say, whether rotten, shivered or broken, or from miscellaneous causes. These reports are then forwarded to the office of the Engineer of Maintenance of Way, where they are consolidated."

TABLE No 6.—Classification of Ties in Track, Atlantic System, Southern Pacific Railroad, June 30th, 1897.

Kind.	Total number.	Percentage of whole.
Cypress	3 051 476	49.6
Untreated heart pine	1 019 435	16.6
Creosoted ties, principally bridge ties.	42 804	0.7
Burnettized sap pine	1 704 012	27.7
California red-wood,	323 371	5.3
Oak	9 214	0.1
Totals	6 150 312	100.0

TABLE No. 7.—Tie Renewals, Atlantic System, Southern Pacific Railroad.

Year.	Miles of track including sidings.	Number of ties renewed of all kinds.	Percentage of removals to total in use.	Average percentage per year for 5 years.
1887	1 931.56	498 581	9.8	.....
1888	1 946.86	589 969	11.5	.....
1889	2 018.66	508 150	9.5	.....
1890	2 025.22	410 153	7.5	.....
1891	2 049.07	497 617	9.2	9.5
1892	2 063.18	495 287	8.8	.....
1893	2 085.32	428 053	7.3	.....
1894	2 128.65	365 393	6.1	.....
1895	2 175.97	430 014	7.0	.....
1896	2 192.87	431 628	7.0	7.24
1897*	2 198.28	231 460	3.8	.....

* For 6 months only, from January 1st, 1897, to June 30th, 1897.

The figures for removals in 1893 include the removals of previous years, separate figures for which were lost.

"It will be observed from Table No. 8 that nearly all the 1887 ties have been removed, whilst an unusually large number of 1888 have also been taken out. Our records for these years show that a large percentage of the removals was due to the ties having been shivered and broken."

This is attributed to the fact that during the first few years the timber was overheated, the temperature in the cylinders being at

TABLE No. 8.—Treated Ties Laid and Renewed, by Years, Atlantic System, Southern Pacific Railroad.

Year Treated.	1887.	1888.	1889.	1890.	1891.	1892.	1893.	1894.	1895.	1896.	1897.	
No. of treated ties laid in same year.	4 144	118 098	196 365	212 677	180 670	247 179	179 229	190 769	190 802	165 783	86 737	
Number of ties treated each year, removed each year. Year removed.												Total removed.
1893	472	3 199	528	69	.....	.....	.....	.....	.....	.....	.....	4 268
1894	1 020	3 602	547	67	8	.....	.....	.....	.....	.....	.....	5 244
1895	1 156	10 665	3 337	940	185	46	269	6	.....	.....	.....	16 604
1896	1 401	16 883	5 545	3 536	936	336	177	18	129	.....	.....	28 961
1897*	95	9 419	3 867	4 424	1 035	444	120	79	1	1	.....	19 485
Total number removed	4 144	43 768	13 824	9 036	2 164	826	566	103	130	1	.....	74 562
Number of years laid in track on July 1st, 1897	10	9	8	7	6	.....	.....	.....	.....	.....	.....	.....
Percentage remaining in track July 1st, 1897	0	63	93	96	98.8	.....	.....	.....	.....	.....	.....	.....

* These figures are for six months only, from January 1st to June 30th, 1897.

times as high as 300° Fahr.; and also to the strength of the solution. The latter has already been commented upon. The average life of the ties treated in 1887, considering the 472 reported as removed in 1893 as actually coming out that year, is 8 years, the amount of error from this assumption being small. Of the later years the average life of the ties treated cannot now be figured, as many of them are still in service. Of the 1888 ties 63% were still in track after 9 years' service; of the 1889 ties, 93% after 8 years' service; of the 1890 ties, 96% after 7 years' service.

It is not claimed that the decrease in the percentage of renewals of ties of all kinds during the past 10 years from an average per year of 9.5% for the years 1887–91, to an average of 7.24% for the years 1892–96, is due solely to the use of the preservative process; but, in the opinion of Mr. Kruttschnitt, that is an important factor in it. He says:

"Many years ago I became convinced that the greatest economies that could be effected in railroad operation would be those effected by

the treatment of timber. From the year 1885 when I assumed charge of the Atlantic System, I have been consistently working to the policy of using nothing but Burnettized or creosoted ties or redwood and cyprus. The California redwood, when protected against crushing, is practically everlasting, and our statistics show the gradual increase in the number of what we might call permanent ties, and a gradual decrease of untreated pine and oak. We hope within the next 3 or 4 years to make the latter disappear entirely. The reduction in the number of ties renewed per mile is due to three causes:

"*First.*—Treatment of timber.

"*Second.*—Reduction in the use of heart pine and increase in the use of redwood and cyprus.

"*Third.*—To tie plating."

Quoting again from Mr. Mahl's report:

"The objection to the Burnettized ties, as we have treated them, is that moisture will dissolve the chloride around the ends and outer surface of the ties, thus leaving an outer shell unpreserved, the timber in the same having resumed its natural condition, which, for the class of ties we use, is poor, indeed. The extent of this washing out of the zinc is dependent upon the annual rainfall, the quality and cross-section of the ballast used and the efficiency of the drainage."

The effects of moisture on the ties in the different sections being merely conjectural, beginning in 1896 the records were so arranged as to permit each roadmaster's district to be studied independently. The results so far obtained can be considered as indicative only, but they are interesting.

The Glidden District has a length of main line of 86 miles, beginning at Houston, Tex., and extending west to Glidden. It runs through a low flat country, and is ballasted with coarse gravel which extends 3 ins. above the ties at the center of the track and slopes so as to leave the ends of the ties free. The ballast is 8 ins. thick under the ties. The average yearly rainfall at Houston is 49 ins., which is a fair allowance for the entire division. A $61\frac{1}{2}$-lb. rail with a $4\frac{1}{4}$-in. base is used.

The San Antonio District joins the preceding at Glidden and ends at San Antonio, Tex. It has a length of main line of 121 miles. The profile is undulating, the line crossing the drainage of the country. A $61\frac{1}{2}$-lb. rail, with a $4\frac{1}{4}$-in. base, is used. The ballast is the same as for the Glidden District, and the roadbed is well drained. The average yearly rainfall at San Antonio is 29.5 ins., and 30 ins. can be assumed as the precipitation for the San Antonio District.

The Del Rio District has a length of main line of 170 miles. The

profile is undulating, and there are some grades of 1 per cent. The roadbed is of standard cross-section, well drained, and is ballasted with coarse gravel. A 50-lb. rail, with a $3\frac{1}{2}$-in. base, is used. The rainfall for the District is given by the Government Report on Climate, for 1896, as between 20 and 30 ins.

The Sanderson District has 138 miles of main line, and runs almost entirely through mountainous country, with maximum grades and curves of 1% and 10°, respectively. The rail is half 60-lb. and half 50-lb. There is no ballast, the ties lying on the natural bed of soil and rock. The rainfall is less than for the Del Rio District, being given in the Government report as from 10 to 20 ins.

The Valentine District has 154 miles of main line, through country of the same character as the Sanderson District. The rail is about half 70-lb. and half 50-lb. The track is ballasted for 42 miles with fine gravel, and is of standard section. The rainfall given is the same as for the Sanderson District, but is probably nearer the lower limit.

The El Paso District has 161 miles of main line through country of the same character as the two preceding sections. The rail is half $61\frac{1}{2}$-lb., and half 50-lb. There is no ballast. The rainfall is light, and about 55 miles of the line is through arid country; probably 8 to 10 ins. precipitation would be an average for the District. The removal of treated ties only, on these various districts, for the 28 months ending April 30th, 1898, is given in Table No. 9.

TABLE No. 9.—Removals of Burnettized Ties and Causes thereof on Six Divisions, Atlantic System, Southern Pacific Railroad, from January, 1896, to May, 1898.

District.	No. of ties removed.	Cause of Removal.						Average life of ties removed.
		Rotten.		Shivered, broken.		Accidental and miscellaneous.		
		No. of.	Per cent.	No. of.	Per cent.	No. of.	Per cent.	
Glidden.....	6 048	5 542	92	506	8	0	0	6 yrs. 9 mos.
San Antonio...	41 691	36 750	88	4 691	11	250	1	8 " 2 "
Del Rio.....	15 564	14 247	92	1 240	8	77	.....	8 " 10 "
Sanderson.....	2 607	1 716	66	145	5	746	29	6 " 8 "
Valentine.....	12 899	10 911	85	1 825	14	163	1	7 " 0 "
El Paso.....	13 397	10 898	81	1 596	12	903	7	7 " 6 "
	92 206	80 064	87	10 003	11	2 139	2	7 yrs. 11 mos.

It should be noted that the average life given in the table is that of the ties removed, and is of little value as indicating the probable average life of a lot of ties unless the ties have been in the track long enough to have covered the life of the most durable; and instead furnishes only a minimum life. To illustrate this, Table No. 10, covering additional data for the first two districts, is given. It will be noted that the average life of the ties removed is increasing steadily. The discrepancies in the figures for the average life in the two tables are due to the first covering 28 months, while in the second the averages are for each year.

TABLE No. 10.—REMOVALS OF BURNETTIZED TIES FROM TWO DISTRICTS, ATLANTIC SYSTEM, SOUTHERN PACIFIC RAILROAD.

Year.	TIES REMOVED FROM TRACK.		AVERAGE LIFE OF TIES REMOVED.	
	Glidden District.	San Antonio District.	Glidden District.	San Antonio District.
1894....	848	4 109	5 yrs. 8 mos.	6 yrs. 2 mos.
1895....	841	10 330	6 " 5 "	6 " 10 "
1896....	1 871	16 904	5 " 11 "	7 " 9 "
1897*...	1 750	8 186	7 " 5 "	8 " 2 "
1898....	3 637	19 986	7 " 0 "	8 " 8 "

* 6 months.

In Table No. 9 the average life on the Sanderson District is much lowered on account of the large number of removals due to accidents and miscellaneous causes. On the Valentine and El Paso Districts the average is also lowered by extensive rail renewals during the last 3 years. Mr. Mahl's opinion is "that the relative difference in life shown to exist between the Glidden, San Antonio and Del Rio districts may be reasonably taken as representing the relative life of ties for those districts." Mr. Mahl states: "With few exceptions the treated ties have been used exclusively in the main line between Houston and El Paso," being composed of the Divisions previously described. By comparing Tables Nos. 8 and 9, it will be seen that the average life of all the ties removed on all the divisions in the 18 months ending July 1st, 1898, was 7 years and 11 months; and that on July 1st, 1897, of all the ties laid 8 years previously, 93% was still in the track.

"The track work on these districts during the period in question consisted by far of only ordinary section labor engaged in the repair

and maintenance of the roadbed and track. For some years past, our repairs of roadbed and track over the entire system have shown gratifying decreases each year, until, for the present year ending June 30th, 1898, these repairs will not exceed $305 per mile of main line for the part west of Houston. This is the lowest cost for repairs of roadbed and track that has ever been obtained, and it is in part attributable to the increased percentage of treated ties in track."

The results of Burnettizing were so satisfactory on the Atlantic System that a plant was built in 1894 for the Pacific system of the Southern Pacific Company, and, as the timber supplies were widely separated, for economy the entire works were placed on wheels, and when a change in location is necessary, the plant forms a train which has successfully traversed 16° curves and 3% grades. It consists of two retorts, 6 ft. in diameter and 114 ft. long, and has a capacity of 2 500 to 3 000 ties in 24 hours. The total number of treated ties in the tracks of the Pacific system, up to the end of 1897, was 1 448 315, all but 74 012 of which were placed subsequent to 1894; consequently no information is yet available as to their life. This particular plant is described in a paper* by W. G. Curtis, M. Am. Soc. C. E., and was built under patents owned by Messrs. Curtis and Isaacs, both of the Southern Pacific Railroad.

A similar plant is now under construction for Mr. O. Chanute, to treat ties for the Chicago and Eastern Illinois Railroad.

Summary and Discussion of Recent American Experience.

From the preceding, the experience of American roads with treated ties may be concluded to be generally favorable. The Atchison, Topeka and Santa Fé Railroad officials, after 12 years' trial on a large scale, believe they are getting from 11 to 12 years' service from mountain pine, having a natural life of about 4 years; while from natural white oak they get but 6 years in heavy main line service; and from cedar 10 years under light service.

The Union Pacific Railroad reports about 9 years for mountain pine and spruce. The Chicago, Rock Island and Pacific Railroad report of the count of ties now in place, indicates a life of at least 8¾ years. The unimpeached counts of removals prior to 1894 show that in that year 84.7% of the ties laid 7 years previously was still in track. The record of the four sections just west of the Missouri River, as here tig-

* *Journal* of the Association of Engineering Societies, July, 1895.

ured, indicates a life of 11 years. The experience of the Pittsburg, Fort Wayne and Chicago Railroad with hemlock and tamarack ties, only 1% having been removed after 7 years; and of the Duluth and Iron Range Railroad, with white and Norway pine and tamarack which, after 7½ years' service and after having been taken up and relaid once, were still in nearly perfect condition; furnish conclusive evidence that the treatment is efficacious, and presumptive evidence that a life of 11 or 12 years may be expected. The discovery of about the same number of treated ties in the tracks of the Rock Island and Peoria Railroad, as were laid in 1891—at which time the last were laid—demonstrates how entirely mistaken were the officers of that road as to the life to be secured from such ties, even under the unfavorable conditions existing as to drainage and ballast.

The Southern Pacific Railroad, operating on the most perishable material, with a life of only 3 years at the best, had 93% of the ties treated and laid in 1889 still in the track after 8 years. The record for the ties treated during the 2 previous years was not as good, for known reasons; however, those laid during 1887 gave 10 years' service, and of the 1888 ties, 63% was still in use 9 years later.

The woods which have been treated and used for a considerable time in regular service are the mountain pines, firs and spruces, hemlock and the sapwood of the yellow pine. The woods used experimentally are the cottonwood, sweet gum, red and black oaks, beech, tamarack and Norway and white pine. Of other timbers which have not been tested, it can be expected that beneficial results will be obtained from treating any wood which will take the treatment well; that is, into which the proper amount of chloride can be injected. This is determined by the physical characteristics of the timber, the size of the sap ducts, the thickness of their walls and the amount of gummy substance contained in the sap. It does not follow, however, that equally good service will be secured from two materials, even if they take treatment equally well. The table on page 217 shows that in the German practice their beech and pine are treated with the same quantity of chloride, but the pine has 3 years' longer life, if both are Burnettized. Treated with tar oil, the beech is apparently the longer lived by 50 per cent.

The German practice is to treat only thoroughly seasoned wood. This is generally considered to be desirable in America also, but seems

to be unattainable. Mr. W. G. Curtis, however, in his paper describing the practice of the Southern Pacific Railroad, Pacific System, already referred to, states that freshly cut ties take the treatment much more readily than those partly seasoned; so that if the ties are freshly cut, they can get six charges through two cylinders in 24 hours; while if partly seasoned, they get but five charges.

"We find the time required varies greatly with the kind of timber and with the time during which the ties have been seasoning. California mountain pine, fir and spruce require less time than Oregon fir, and all timbers are more readily treated when freshly cut. An Oregon fir tie seasoned in the air for 2 years will take double the time for the treatment required for one freshly cut. Occasionally a close-grained, well-seasoned tie will not receive the preservative at all, the fluid penetrating into the sides only about $\frac{1}{2}$ in."

While this statement is probably correct for the particular timber treated, it is entirely contrary to the practice at other places, both here and abroad. As already quoted at the Houston works of the Southern Pacific Railroad, "if green ties and ties which have been seasoned for 3 to 6 months were subjected to the same treatment, the percentage of absorption of the seasoned ties would be much greater than the absorption of the green ties." The Chicago Tie Preserving Company finds it impossible to get satisfactory work with freshly cut ties, whether of hemlock, beech or tamarack. During 1898 this company ceased operating entirely for nearly 2 months, and piled up 70 000 ties in the yard at considerable expense for rehandling, to permit them to season sufficiently to take the preservative. These peculiarities of timber, however, are not necessarily dangerous. It is simply necessary to fit the treatment to the material; and emphasizes two things: That such a cost should be allowed for treatment as to permit of occasional exceptional expense without loss, and that persons entrusted with such work should be above sacrificing the quality of their work for a small immediate gain.

The Chicago Tie Preserving Company has very kindly given access to all its notes and records, both of routine and experimental work; and by request has submitted the following statement:

"This company has made hundreds of experiments upon thousands of ties. These were tried partly by weighing individual ties before and after treatment in the regular runs, to ascertain the absorption; and partly, since 1894, by special treatments in an experimental retort, holding three ties, wherein various methods could be tested without

interfering with the regular runs. The objects in the latter were twofold:

"*First.*—To ascertain by what modification the largest quantity of chloride of zinc could be injected without making the timber brittle. This has been attained, the amount of antiseptic absorbed since 1896 having averaged double that injected in 1886, with what results remains to be seen, while not a tie has been reported as having broken in the track.

"*Second.*—To ascertain whether the ties were in proper condition to treat, and the idiosyncrasies of various woods.

"As regards the conditions, it has been found that ties delivered early in the spring require to be treated differently than those arriving in the late summer. Sometimes the former are quite unfit and must be placed upon the ground to season. This should preferably be done in cribbed piles (when there is space enough) to allow the air to circulate between the ties. The same has to be sometimes done in the autumn, when the ties have been rafted or floated out to the vessels.

"Refractory lots of hemlocks are piled upon the ground, and a few tested month by month to ascertain their condition, It is found that they gradually improve in receptivity, so that in 6 months they absorb two to three times as much as upon their first arrival. This agrees with European experience, where ties are seldom treated less than a year after being cut, but it is the reverse of the experience of Mr. W. G. Curtis, on the Southern Pacific System in California and Oregon," as quoted above. "The same claim was made by Mr. O. Thilmany, who stated: 'We prefer green timber; dry timber has to be steamed longer. Only live and sound timber can be preserved.'

"Perhaps these puzzling contradictions are due to differences in the character of the woods treated, or in the time of cutting.

"It is well recognized that natural woods are more lasting if cut when the sap is down. Summer-cut hemlock ties tested repeatedly by the Chicago Tie Preserving Company only absorbed about half as much as those winter-cut, and it was also found that some gum and pine ties cut in Virginia, North Carolina and Georgia in September, absorbed from two to eight times as much as ties from the same woods cut in April.

"It is most desirable, before treating a tie, to know when it was cut, and an ideal condition would be to have the ties identified as to the time of their cutting, and treated in the same order.

"After treatment all ties should be allowed to dry, as is always done in Europe. Several lots were treated twice in Chicago, having absorbed in the first treatment but about 14% of their weight. All of this gain they lost in 5 months of seasoning, and 12% more; and were then in a proper condition for a second treatment.

"As regards the idiosyncrasies of various woods, it has been ascertained that hemlock and beech are much more refractory than pine, principally early in the season; that white and burr oak should not be Burnettized, but that black oak, red oak and water oak take the treatment very well, and will outlast Colorado pine. Cedar is hardened by the process, but not enough to pay for the cost of injection. Tamarack, like spruce, is an uncertain and variable wood. Some ties cut at the same time, from the same forest, were found to vary as 2 is to 1 in average absorption. This was attributed to the varying amount of resin in various trees, and after many experiments it was learned to distinguish this difference by the eye, but, as it was impracticable to sort the trees before cutting them down, the injection of tamarack was finally abandoned altogether.

"There were also great differences in the absorption of individual ties of the same age and same species of wood. In extreme cases some will take eight times as much as their neighbors, and it becomes a serious problem how to regulate the injection. It will not do to treat all kinds of wood alike, and, when a new species comes to the works, many experiments have to be tried before it is known how it is best to deal with it.

"The operations of the works are, of course, controlled by an elaborate system of measurements and accounts, so that it is known accurately how much of the antiseptics are absorbed by each run of, say, 400 ties, but it is not known how much each individual tie has taken, save in special experiments. Changes are in contemplation to circumscribe this uncertainty, and in this, as well as in other details, we shall have to profit by the experience of the Europeans, who have been preserving ties for about 50 years, and have materially improved upon their original methods."

In examining the records of tie treatments it is necessary to recognize the fact that the life of the exceptional tie, whether long or short, is of no value, except as a curiosity, and as containing a possible lesson in detail. As an instance, a tie was exhibited in the Paris Exposition of 1889 which had lasted 46 years in the track of one of the French railways. It had been prepared by the sulphate of copper (Boucherie) process, but nevertheless this process has been generally abandoned in favor of others. So far as the value of a treatment itself is concerned, such records of individual ties may be worse than useless, leading to an expectation of length of life which cannot be attained by the average, or discouraging the investigator without reason. The valuable thing is the average durability, and by this only can any results of real value be obtained.

The author has been impressed, while reading the statements of 10 years ago, with the fact that the advocates of preservative methods claimed too much. The life of treated ties was unhesitatingly put at 15 years, a limit which the occasional tie has reached, and which indeed, under favorable circumstances, the foregoing records show to have been reached by a considerable number of ties, and to which, with future treatment, as the operators become more skilful in the selection of material and its application, perhaps the average life may approach. The record of commercial use in the United States has failed to reach any such period, and it is entirely unnecessary that it should, to make the treatment desirable and economical.

The author believes his investigation has demonstrated that the cheaper woods, hitherto unused, are made fully as durable or more durable than white and burr oak, by Burnettizing; and this fact is sufficient to make the adoption of the treatment desirable for many roads, if indeed it may not be said for American roads generally.

One point this inquiry has not solved is the relative advantages of the zinc-tannin or Wellhouse process, and the old method of Burnettizing, using nothing but the zinc chloride. This hinges upon two points: The solubility of the chloride when deposited in the wood, and the efficiency of the chemical action between the glue and the tannin, required to form the particles of leather which are supposed to plug up the ducts. As to the first, there is general evidence that part of the chloride can be washed out of the timber, although it is not easily done. Mr. J. D. Isaacs, of the Maintenance of Way Department of the Southern Pacific Company, in a paper* states:

"The author thinks this objection is greatly exaggerated. He has attempted to leach this chloride out of timber with running water, but could extract only an insignificant proportion, even when the timber was in fine shavings. All the chloride could not be taken out of chips from treated timber by boiling in repeated changes of water. If leaching takes place, it is certainly very slowly."

The above applies to wood treated with the chloride only. Unfortunately, this railroad was unable to furnish any chemical analyses of treated timber which had been in service for a number of years. That the chloride is gradually leached out of material treated by the zinc-tannin process is probably true. A number of analyses of timber after years of exposure shows considerably smaller quantities of zinc

*Thirteenth Report of the California State Mineralogist. 1896.

than it is believed they originally contained. One such instance has been given in connection with the Union Pacific ties, where only about one-fourth as much of the chloride was found as was generally injected. The evidence at hand would be absolutely conclusive if it were possible to know how much the ties which were analyzed contained originally; but this is not the case, as the amount of absorption of a retort full of ties is as near an individual record as it is now possible to go; and it is possible that the tie analyzed might have been one which was refractory and took very little when treated. If such had been the case, however, the tie would probably have been of short life; and it is safe to conclude that after a certain number of years, depending upon the rainfall and the efficiency of the track drainage, the amount of chloride remaining in a tie will be so reduced as to be of no further value, whether the treatment be with zinc alone or in combination with the glue and tannin.

If the claims for the formation of the artificial leather by the tannin treatment are correct, and it is understood that this is verified by microscopic examination, it is probable that the leaching is delayed. That the treatment does not prevent the passage of moisture into the wood is evidenced by the rapidity with which the treated timber seasons, as well as by the loss of the zinc shown by the analyses. The results in future on the Atchison, Topeka and Santa Fé Railroad may determine the relative advantages of these two systems, as there are large numbers of ties now in use there treated by each. To insure the greatest success by either, however, it is believed that the injection of large amounts of chloride must be practiced, and, generally, this can be done only by using timber at least partly seasoned; and the subsequent partial seasoning before being placed in track.

Granting that certain woods may take the antiseptic in sufficient quantities even if treated green, so that it may not be accurate to say that it is necessary to good work that at least partly seasoned wood should be treated, there are certain other conditions which can be said to be essential to good work. These are pure zinc, pure acid, the absence of any free acid in the chloride solution, the use of a heat not exceeding 250° in the retorts, and sound, live timber to operate on. To treat dead or dozy wood is to waste time and chemicals. Chloride of zinc apparently has no power to stop decay which has already begun, and it is doubtful if any treatment is better in that

respect. To these may be added faithfulness and integrity of a high order on the part of those operating the works.

The output and profits of a plant can be very largely increased by the lack of the last, and the results of such lack will not be apparent until years afterward.

To make the later identification of the ties certain, so that the life realized can be determined, it is believed that nothing will prove as satisfactory as the use of dating nails, which are nails with large heads in which is stamped the current year, and which are driven in the center of the top surface of the tie when laid. The cost of such nails is but small, and they are more accessible and certain than any substitute.

Foreign Practice.

The author regrets that he cannot furnish very much on the foreign practice of recent date. In France and Great Britain, the creosote process is used almost exclusively. The Burnettized timber was apparently not satisfactory. Creosote is comparatively cheap, and the increased life of ties so treated is considered as worth the additional cost over Burnettized ones.

The roads in the German Railroad Union, however, use very generally, either zinc chloride or a combination of the chloride and creosote, as well as creosote alone. Impregnation with the chloride alone has decreased while that with the mixture of chloride and creosote has largely increased. Beginning in 1895 all the pine ties used on the Prussian State railroads were treated by the zinc creosote process. This consists of the injection of the same amount of zinc chloride, with which is mixed creosote, in the amount of $1\frac{1}{4}$ lbs. to the cubic foot of timber; the mixing being done by a peculiar process. It is claimed that the results of this treatment are much superior to those with zinc chloride alone. In 1885, the Ruetgers Works furnished one road with 171 000 pine ties treated with zinc creosote, on a 9-year guarantee. In 1894, at the end of the 9 years, only twenty-nine had become unfit for use and none of these was rotten. Mr. Chanute is about to go to Germany to investigate this process, and it is hoped that he may be able to contribute the necessary data to the discussion of this paper, so that the value of the treatment may be determined. The additional cost over the regular Burnettizing process varies with the price of creosote, but is about 4 cents per tie. The zinc tannin process has never been used outside of the United States.

The Ruetgers Works treat ties by Burnettizing on the following specifications: The solution of zinc chloride is to be 3° Beaumé strength at a temperature of 14° Réaumur. Ties and timber are to be treated until the absorption of seasoned timber is for

Pine wood.....19 lbs. per cubic foot.
Oak " ..... 6½ " "
Beech " .....19 " "

" If the amount of liquid required cannot be injected in the manner prescribed because the wood is too green or is exceptionally dense, the liquid is to be strengthened in such a manner that the proper amount of water-free chloride of zinc is taken up as prescribed, namely, that to be contained in a 3° Beaumé liquid.

" From this it follows that when, for instance, only 200 kilograms can be introduced into 1 cu. m. of pine wood, the chloride of zinc bath must be made 4½° Beaumé."

The German report on railroads previously referred to gives certain tables from which the following are obtained:

TABLE No. 11.—Costs and Results of Wood Preserving for the Union of German Railroads for 1896.

Kind of tie.	Cost of crude tie.	Treated with Chloride of Zinc.					Treated with Tar Oil—Creosoted.				
		Absorption in pounds.	Cost of treatment.	Total cost.	Average duration, in years.	Cost per year.	Absorption, in pounds.	Cost of treatment.	Total cost.	Average duration, in years.	Cost per year.
			Cts.			Cts.		Cts.			Cts.
Oak....	$1.49	24.2	13	$1.62	15	10.8	15.4	21	$1.70	24	7.1
							24.3	29	1.78	28	6.3
Beech..	1.01	34	15	1.16	9	13.0	66	50	1.51	30	5.0
							79.2	59	1.60	34	4.7
Pine....	0.84	34	16	1.00	12	8.3	50.6	43	1.27	20	6.3
							79.2	57	1.41	23	6.1

This is based on a tie 6¼ ins. × 10 ins. × 8 ft. 10 ins.

From Table No. 12 it will be seen that in 1896 about 60% of all the wood ties in the tracks was impregnated in some way; and that all of the increase in number of ties in track over 1894 was in the treated ties. Unfortunately, it is impossible to determine the number treated by each process in use by these roads, and the figures are of value

only as indicating the general use of treated timber in preference to the crude.

As a matter of interest, a line is added showing the amount of track laid with metal ties or sleepers.

TABLE No. 12.—Statistical Report of the Railroads of the Union of German Railroad Administrations for the Fiscal Years 1896, 1895, 1894.

Designation of railroads.	Year.	Abstract of Wood Ties, Laid in Track. Number of wood ties. Oak.	Beech.	Larch.	Pine.	Total.	Of the ties there are: Impregnated.	Crude.	Length of track laid in metal permanent way. Kilometers.
German, State and private roads	1896	28 279 084	3 248 882	92 585	36 947 827	68 568 378	59 834 087	8 734 291	22 805
	1895	28 635 001	3 283 867	91 924	34 775 635	66 786 427	57 626 304	9 160 123	22 102
	1894	29 353 375	3 132 261	102 936	32 241 230	64 829 802	54 041 505	10 788 297	20 868
Luxemburg, Netherlands and other Union roads.	1896	8 769 969	49 551	.....	541 285	12 099 368	1 718 815	6 730 887	495
	1895	8 281 293	48 392	.....	538 330	11 566 538	1 597 308	6 360 733	497
	1894	7 916 896	39 083	.....	552 105	11 200 509	1 552 809	6 046 430	495
Austria, Hungary and Bosnia roads	1896	31 533 486	5 406 009	4 066 135	6 144 072	47 149 702	18 197 013	28 952 689	404
	1895	30 192 026	5 437 128	3 913 322	6 056 466	45 598 942	17 755 877	27 843 065	368
	1894	29 286 099	5 797 985	3 581 179	5 778 177	44 443 440	16 882 687	27 560 753	366
Totals of all roads	1896	68 582 539	8 704 442	4 158 720	43 633 184	127 817 448	79 749 915	44 417 867	23 704
	1895	67 108 320	8 769 387	4 005 246	41 370 431	123 951 907	76 979 489	43 363 921	22 967
	1894	66 556 370	8 969 329	3 684 115	38 571 512	120 473 751	72 477 001	44 395 480	21 858

Cost of Treatment.

It is not proposed to enter into any discussion of the economy of, or the savings to be effected by, the use of treated ties. This is something which will vary with almost every road, being governed by the cost of white oak ties, cost of the inferior tie to be treated and location. It has been attempted to determine what can be expected from the treatment and with the cost thereof given below, it can readily be determined which is the cheaper—a white oak or a treated tie, and, in the author's judgment, that is what the question resolves itself into.

Apparently the evidence is conclusive that a treated tie is superior to a white oak; but, if only equal to it, on the basis of cost in most locations the treated tie will prove the more desirable.

The cost of Burnettizing sawed pine ties 6 ins. x 8 ins. x 8 ft. on the Southern Pacific Railroad in 1893, was about 10 cents each; while in 1897

it was a little over 6 cents. This includes nothing for interest on the investment, or depreciation; however it is for a very small tie, containing only $2\frac{2}{3}$ cu. ft.; and for an amount of zinc chloride only about one-half that required by the German practice. On the basis of the cost in 1897 (which amounts to $2\frac{1}{4}$ cents per cubic foot), the use of the standard German quantity to be injected and a hewn tie of $3\frac{1}{2}$ cu. ft., this cost would be about 12 cents per tie, or $4\frac{1}{2}$ cents per cubic foot, including nothing for interest or depreciation.

On the Atchison, Topeka and Santa Fé Railroad the cost of the zinc tannin treatment, when the works were first built, was about 15 cents; in 1892 it was about 14 cents and 13 cents for Burnettizing (zinc chloride only). In 1897 the cost (zinc-tannin) was reported as 11.6 cents; no interest or depreciation being included. It is understood that the charge made by the Chicago Tie Preserving Company is from 16 to 20 cents for the zinc-tannin treatment, covering, of course, interest and depreciation as well as profit.

The contract prices in Germany for Burnettizing are: For pine and beech, 5 cents per cubic foot, and for oak, 4 cents; for treating with zinc-creosote, 6 cents for beech and pine, and 5 cents for oak; for creosoting, 15 cents for beech and pine, and 9 cents for oak. In creosoting, the amount of creosote per cubic foot is 12 lbs. for pine, 15 lbs. for beech, and $4\frac{1}{2}$ lbs. for oak. It is understood that the oak referred to corresponds to American white oak and not to the American red and black oaks, which will absorb as much as either pine or beech.

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ON THE THEORY OF CONCRETE.

By George W. Rafter, M. Am. Soc. C. E.

To be Presented June 7th, 1899.

While employed on the original Genesee storage investigation in 1893, the author made a study of the available material for the construction of a high dam on the Genesee River, near Mount Morris, N. Y., the study including the fabrication of 173 blocks of concrete of cubes of 1 ft. each; the cement used, including Cumming's Natural Buffalo cement, Newman's Akron, Bangs and Gaynor's Fayetteville, Norton's Rosendale, Millem's Wayland Portland and Saylor's Portland.

Tests were made, at the same time, of the voids in the local sand and in the different stones used as aggregates, and also of the tensile strength of the several cements. The results of these tests may be found in Appendix E of the Annual Report of the State Engineer and Surveyor of New York, for the fiscal year ending September 30th, 1894.

In 1896 further tests were made, which included the fabrication of 544 additional concrete foot-cubes, the cements tested including Whitmore, Rauber and Vicinus' Genesee brand, Millem's Wayland, the Glens Falls Portland Cement Company's Iron-Clad brand, the Empire Portland Cement Company's Empire brand, the Silica Portland

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in the volumes of *Transactions*.

Cement Company's Champion brand, Cumming's Buffalo Portland and Cumming's Buffalo Natural. The detail of this second set of tests may be found in the Appendix to the Annual Report of the State Engineer and Surveyor of New York, for the fiscal year ending September 30th, 1897, to which reference may be made.

Tables Nos. 1 and 2 embody certain results of the second series of tests made in 1896, which, to a large extent, are not included in the report to the State Engineer, and which have seemed to the author to be of possible interest to the members of the Society. They are, therefore, submitted for comment and criticism.

The following explanations are condensed from the author's report to the State Engineer on these tests.

The leading ideas of the tests made in 1896 were: (1) To determine the strength of concretes fabricated with either dry mortars, plastic mortars or mortars with an excess of water; (2) to determine the effect on concrete of exposure to the weather; and (3) to determine the relative value for use in concrete of the Portland cements manufactured in the State of New York. The various qualities of mortar used are designated by the words "dry," "plastic" and "excess." In the dry blocks the mortar was only a little more moist than damp earth. In the plastic blocks the mortar was of the ordinary consistency used by masons. In the excess blocks the water was so far in excess that the concrete quaked like liver, under moderate ramming.

The serial number of Table No. 1 represents a complete series as designated by the Arabic numeral and the letters "a," "b" and "c" of Table No. 1 of the concrete report.* Thus, by way of illustration, Series 1 of Table No. 1 is a combination of blocks 1, 1a, 1b and 1c or Table No. 1 of the report; Series 2 of Table No. 1 represents blocks 2, 2a, 2b and 2c of Table No. 1 of the report, and so on. The headings of the several columns of Table No. 1 are self-explanatory. The blocks were broken on the large Government testing machine at the Watertown Arsenal.

* On referring to Table No. 1 of the report on concrete in the Annual Report of the State Engineer and Surveyor of New York, for 1897, it will be observed that the blocks were made in series of four. Those designated by the Arabic numeral were placed in water when taken from the molds, and allowed to remain until December 1st, 1896, when they were buried in sand, where they remained until taken out for shipment January 10th and 11th, 1898. Blocks marked with the Arabic numeral and the letter "a" stood in a cool cellar from the time of taking from the molds until shipped. Blocks marked with the Arabic numeral and the letter "b" were placed in open air when taken from the molds, and exposed to the weather until shipped. Blocks marked with the Arabic numeral and the letter "c" were placed in open weather when taken from the molds, covered with burlap and wet with water several times a day until November 1st, 1896, after which time they took the weather as it came, until shipped.

The formulas by which the quantities in columns (14), (17), (18) and (19) have been calculated, are as follows:

FORMULA FOR COLUMN (14).

Dry cement used in batch of concrete. : Mortar used in batch of concrete. :: Dry cement used in mortar, : Cubic feet of mortar made;

or, Column (14), : Column (11), : : Column (7), : Column (9).

FORMULA FOR COLUMN (17).

Number volumes mortar in concrete for one of cement, : One volume of cement, :: Mortar originally made, : Cement used in batch of mortar;

or, Column (17), : 1, : : Column (9), : Column (7).

FORMULA FOR COLUMN (18).

Number volumes stone used in concrete for one of cement, : One volume of cement. :: Stone used in batch of concrete, : Dry cement used in batch of concrete;

or, Column (18), : 1, : : Column (12), : Column (14).

FORMULA FOR COLUMN (19).

Number volumes concrete made for one volume of cement used, : One volume of cement, :: Concrete actually made, : Cement used in batch of concrete;

or, Column (19), : 1, : : Column (13), : Column (14).

In reference to column (10) of Table No. 1 the author holds that an erroneous nomenclature has considerably obscured the theory and practice of concrete work, so much so that very few contractors and engineers have other than exceedingly general ideas as to the real theory of what is in effect a very simple matter, there being, indeed, very little necessary except to insure the filling of the voids in the aggregate material with mortar. In the author's opinion, therefore, the proper form of expressing concrete relations is not by parts 1, 2, 5; 1, 3, 7; etc., but rather to express the volume of mortar as a percentage of the volume of stone, the percentage figure used being large enough to represent a little more than the voids in the stone after thorough ramming. The chief object of this paper, therefore, is to point out: (1) The relation existing between the old nomenclature and the new, as proposed by the author, and, (2) to present a brief *résumé* of the results given in detail in the report to the State Engineer and Surveyor.

In Table No. 2 the series have been arranged for the different proportions of cement experimented upon with reference to the consistency

of the mortar, the proportion of sand and cement and the percentage of mortar actually used in each series. Thus, in the first part of Table No. 2 we find compared the Genesee, Wayland and Champion cements for dry mortars with the proportion of sand to cement 1 to 1 and the mortar 33% of the aggregate. In the second division of Table No. 2 the same cements are compared with reference to plastic mortars, other conditions remaining the same; and in the third division they are compared with reference to excess mortars, other conditions remaining the same, and so on, through this table to the summary.

Column 1 of Table No. 2 gives the serial number, the figures there appearing corresponding not only to Table No. 1 of this paper, but also to Table No. 1 of the report to the State Engineer and Surveyor; columns (3), (17), (18), (19) and (20) of this table are the same as the correspondingly numbered columns of Table No. 1 of this paper. Column A of Table No. 2 gives the percentage of shrinkage due to ramming. Thus, column (18) represents the actual quantity of stone used, and column (19) the actual quantity of concrete obtained from mortar and stone as per columns (17) and (18). In the case of the Genesee, Wayland and Champion brands, 1 to 1 mortar, 33% in the aggregate was tried; this shrinkage is found to be, as a mean, 9.3%; for the same cements in plastic mortar, other conditions the same, the shrinkage is 9.1%, and for excess mortars, 9.2 per cent. Broadly, it may be said, therefore, that for 1 to 1 mortars, using 33% of the aggregate, with through ramming, about 9% shrinkage may be expected. In the same way with 1 to 1 mortars, 40% of the aggregate, about 7% shrinkage may be expected. Similar relations, which may be traced through the entire series of tests, are conveniently summarized in Table No. 3.

It will be noticed in studying Table No. 2, that a few of the results are somewhat irregular. On this point the author can only say that very thorough precautions were taken to have the results reasonably accurate. Probably the irregular results are largely due to occasional variations in the quality of the broken stone used for these tests. There were also some variations in the sand, the weight per cubic foot varying from about 86.5 to 93.5 pounds. So far as the author can see, there is no way of avoiding such variations except by determining the voids in the aggregate material for every series of blocks and by actually drying the sand to a uniform consistency. It may be pointed out, however, that such procedure would defeat the

chief object for which these tests were made, namely, to work out methods of procedure which could be conveniently applied in practice without unduly adding to the cost of concrete. From this point of view the author considers the tests here discussed as exceedingly satisfactory. They are in the fullest sense practical results which may be carried out on work by anybody who will give the theory of concrete moderate study.

It may be further pointed out that columns (15) to (19) of Table No. 1 furnish ready means of computing the proportionate parts of the ingredients for any given volume of concrete.

Many other details could be given, but inasmuch as the object of this paper is to merely call attention to the real points at issue in concrete fabrication and not to repeat statements already in print in the place cited, all details other than those actually necessary for an understanding of the tables of this paper have been omitted. The results are, therefore, submitted in the hope that they may finally lead to more rational concrete practice.

TABLE No. 1.—Data of Concrete Tests.

		Mortar.							Concrete.				Proportional Parts by Volume (as per preceding data).					
(1)	Serial number.																	
(2)	Consistency of mortar.																	
(3)	Brand of cement.																	
(4)	Parts of cement by volume.																	
(5)	Parts of sand by volume.																	
(6)	Water used, in pounds per cubic foot of mortar.																	
(7)	Cement actually used, in cubic feet.																	
(8)	Sand actually used, in cubic feet.																	
(9)	Mortar actually made from water, cement and sand, as per columns (6), (7) and (8).																	
(10)	Stone: Ratio to mortar. Percentage.																	
(11)	Mortar actually used in making the quantities of concrete shown in column (13). Cubic feet.																	
(12)	Stone actually used. Cubic feet.																	
(13)	Concrete actually made from mortar and stone as per columns (11) and (12). Cubic feet.																	
(14)	Dry cement actually used in batch of concrete as per column (13). Cubic feet.																	
(15)	Cement.																	
(16)	Sand.																	
(17)	Mortar.																	
(18)	Stone.																	
(19)	Concrete.																	
(20)	Mean ultimate strength of each series of blocks in pounds per square inch.																	

TABLE No. 1—(*Concluded*).

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
70	Dry.	Ironclad.	1	4	21.3	1 1/16	2	2.06	40	1.8	4.5	4.17	0.43	1	4	4.12	10.46	9.70	1 652
71	Plastic.	"	1	2	.....	1	2	2.69	40	1.8	4.5	4.17	0.67	1	2	2.69	6.71	6.22	2 994
72	"	"	1	3	.....	2	2	2.25	40	1.8	4.5	4.17	0.54	1	3	3.38	8.38	7.70	1 839
73	"	"	1	4	.....	2	2	2.23	40	1.8	4.5	.....	0 40	1	4	4.46	11.25	.....	1 544
74	Excess.	"	1	2	.....	1	2	2.62	40	1.8	4.5	4.17	0.69	1	2	2.62	6.51	6.03	2 866
75	"	"	1	3	.....	2	2	2.27	40	1.8	4.5	4.17	0.53	1	3	3.40	8.50	7.86	2 083
76	"	"	1	4	.....	2	2	2.27	40	1.8	4.5	4.17	0.40	1	4	4.54	11.25	10.43	1 508
77	Dry.	Empire.	1	2	20.6	1 1/2	2	1.87	33	1.5	4.5	4.08	0.60	1	2	2.49	7.50	6.80	3 265
78	"	"	1	3	20.0	2	2	2.17	33	1.5	4.5	4.08	0.46	1	3	3.25	9.78	8.62	1 992
79	"	"	1	4	18.4	2	2	2.13	33	1.5	4.5	4.08	0.35	1	4	4.26	12.85	11.65	1 666
80	Plastic.	"	1	2	.....	1 1/2	2	1.79	33	1.5	4.5	4.08	0.62	1	2	2.39	6.71	6.09	2 529
81	"	"	1	3	.....	2	2	2.29	33	1.5	4.5	4.08	0.44	1	3	3.44	10.23	9.26	2 014
82	"	"	1	4	.....	2	2	2.19	33	1.5	4.5	4.08	0.34	1	4	4.38	13.24	12.00	1 523
83	Excess.	"	1	2	21.7	1 1/2	2	1.97	33	1.5	4.5	4.08	0.60	1	2	2.63	7.50	6.80	2 899
84	"	"	1	3	21.7	2	2	2.29	33	1.5	4.5	4.08	0.44	1	3	3.44	10.23	9.26	1 898
85	"	"	1	4	22.3	2	2	2.21	33	1.5	4.5	4.08	0.34	1	4	4.42	13.24	12.00	1 488
86	Dry.	"	1	2	20.0	1	2	2.52	40	1.8	4.5	4.17	0.71	1	2	2.52	6.33	5.87	3 413
87	"	"	1	3	18.2	2	2	2.15	40	1.8	4.5	4.17	0.56	1	3	3.23	8.03	7.43	2 376
88	"	"	1	4	18.1	2	2	2.12	40	1.8	4.5	4.17	0.42	1	4	4.24	10.71	9.91	1 806
89	Plastic.	"	1	2	20.7	1	2	2.56	40	1.8	4.5	4.17	0.70	1	2	2.56	6.43	5.96	3 101
90	"	"	1	3	20.9	2	2	2.21	40	1.8	4.5	4.20	0.54	1	3	3.32	8.33	7.75	2 123
91	"	"	1	4	20.5	2	2	2.13	40	1.8	4.5	4.17	0.42	1	4	4.26	10.71	9.93	1 726
92	Excess.	"	1	2	23.4	1	2	2.56	40	1.8	4.5	4.17	0.70	1	2	2.56	6.43	5.96	2 729
93	"	"	1	3	23.7	2	2	2.29	40	1.8	4.5	4.17	0.52	1	3	3.44	8.58	8.00	1 913
94	"	"	1	4	22.9	2	2	2.17	40	1.8	4.5	.....	0.41	1	4	4.34	10.97	.....	1 472
95	Dry.	Champion.	1	1	19.1	1 1/16	1 1/16	2.46	33	1.5	4.5	4.08	0.90	1	1	1.64	5.00	4.53	2 659
96	"	"	1	2	18.8	1	2	2.37	33	1.5	4.5	4.17	0.63	1	2	2.37	7.14	6.61	2 289
97	"	"	1	3	17.3	2	2	2.13	33	1.5	4.5	4.17	0.47	1	3	3.17	9.59	8.90	1 401
98	Plastic.	"	1	1	21.3	1 1/16	1 1/16	2.37	33	1.5	4.5	4.08	0.94	1	1	1.58	4.79	4.35	2 553
99	"	"	1	2	20.0	1	2	2.42	33	1.5	4.5	4.08	0.62	1	2	2.42	7.25	6.58	2 146
100	"	"	1	3	20.0	2	2	2.17	33	1.5	4.5	4.08	0.46	1	3	3.26	9.78	8.87	1 337
101	Excess.	"	1	1	22.2	1 1/16	1 1/16	2.13	33	1.5	4.5	4.08	0.88	1	1	1.70	5.11	4.64	2 279
102	"	"	1	2	21.4	1	2	2.48	33	1.5	4.5	4.08	0.60	1	2	2.48	7.50	6.80	1 862
103	"	"	1	3	24.2	2	2	2.00	33	1.5	4.5	4.08	0.50	1	3	3.00	9.00	8.16	1 168
104	Dry.	"	1	1	18.5	1 1/4	1 1/4	1.96	40	1.8	4.5	4.25	1.15	1	1	1.57	3.90	3.70	3 125
105	"	"	1	2	19.4	1	2	2.06	40	1.8	4.5	4.25	0.87	1	2	2.06	5.18	4.90	2 204
106	"	"	1	3	19.0	2	2	2.13	40	1.8	4.5	4.17	0.56	1	3	3.20	8.03	7.44	1 400
107	Plastic.	"	1	1	22.1	1 1/4	1 1/4	2.08	40	1.8	4.5	4.08	1.08	1	1	1.66	4.16	3.77	2 603
108	"	"	1	2	.....	1	2	2.44	40	1.8	4.5	4.17	0.74	1	2	2.44	6.08	5.62	2 037
109	"	"	1	3	20.0	2	2	2.19	40	1.8	4.5	4.17	0.55	1	3	3.29	8.18	7.58	1 193
110	Excess.	"	1	1	22.3	1 1/4	1 1/4	2.10	40	1.8	4.5	4.17	1.07	1	1	1.68	4.20	4.00	2 196
111	"	"	1	2	21.0	1	2	2.48	40	1.8	4.5	4.17	0.73	1	2	2.48	6.18	5.71	1 711
112	"	"	1	3	22.2	2	2	2.29	40	1.8	4.5	.....	0.52	1	3	3.44	8.65	.....	1 127
113	Plastic.	Buffalo Por'd	1	2	19.6	1	2	2.54	40	1.8	4.5	4.17	0.70	1	2	2.54	6.43	5.96	1 717
114	"	"	1	3	20.0	2	2	2.19	40	1.8	4.5	4.17	0.55	1	3	3.29	8.18	7.58	958

TABLE No. 1—(*Continued*).

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
25	Excess.	Genesee.	1	1	24.0	$1\frac{1}{2}$	$1\frac{1}{2}$	2.50	40	1.8	4.5	4.17	1.08	1	1	1.67	4.16	3.85	3 191
26	"	"	1	2	22.9	1	2	2.75	40	1.8	4.5	4.08	0.66	1	2	2.75	6.82	6.19	2 544
27	"	"	1	3	20.8	$\frac{2}{3}$	2	2.33	40	1.8	4.5	4.08	0.51	1	3	3.50	8.80	8.00	2 107
28	"	"	1	4	21.0	$\frac{1}{2}$	2	2.08	40	1.8	4.5	.....	0.43	1	4	4.16	10.40	.....	1 846
29	Dry.	Wayland.	1	1	.....	$1\frac{1}{4}$	$1\frac{1}{4}$	1.87	33	1.5	4.5	4.08	1.00	1	1	1.50	4.50	4.08	4 267
30	"	"	1	2	.....	$\frac{3}{4}$	$1\frac{1}{2}$	1.64	33	1.5	4.5	4.17	0.68	1	2	2.19	6.62	6.12	2 888
31	"	"	1	3	18.8	$\frac{2}{3}$	2	2.08	33	1.5	4.5	4 17	0.48	1	3	3.12	9.38	8.69	2 056
32	"	"	1	4	17.7	$\frac{1}{2}$	2	2.12	33	1·5	4.5	4.08	0.35	1	4	4.24	12.86	11.66	1 810
33	"	"	1	5	17.1	$\frac{1}{2}$	$2\frac{1}{2}$	2.58	33	1.5	4.5	4.17	0.29	1	5	5.16	15.41	14.39	1 537
34	Plastic.	"	1	1	19.0	1	1	2.17	33	1.5	4.5	4.08	0.69	1	1	2.17	6.50	5.90	4 072
35	"	"	1	2	21.3	$\frac{3}{4}$	$1\frac{1}{2}$	1.83	33	1.5	4.5	4.12	0.61	1	2	2.44	7.39	6.75	2 777
36	"	"	1	3	20.0	$\frac{2}{3}$	2	2.12	33	1.5	4.5	4.08	0.47	1	3	3.18	9.57	8.68	2 207
37	"	"	1	4	21.2	$\frac{1}{2}$	2	2.12	33	1.5	4.5	4.08	0.35	1	4	4.24	12.80	11.34	1 600
38	"	"	1	5	22.0	$\frac{1}{2}$	$2\frac{1}{2}$	2.50	33	1.5	4.5	4.08	0.30	1	5	5.00	15.00	13 60	1 586
39	Excess.	"	1	1	24.3	1	1	1.75	33	1.5	4.5	4.08	0.86	1	1	1.75	5.23	4.75	3 764
40	"	"	1	2	22.1	1	2	2.04	33	1.5	4.5	4.08	0.73	1	2	2.04	6.23	5.55	2 847
41	"	"	1	3	22.4	$\frac{2}{3}$	2	2.08	33	1.5	4.5	4.08	0.48	1	3	3.12	9.37	8.50	1 723
42	"	"	1	4	22.5	$\frac{1}{2}$	2	2.12	33	1.5	4.5	4.08	0.35	1	4	4.24	12.71	11.70	1 767
43	"	"	1	5	23.0	$\frac{1}{2}$	$2\frac{1}{2}$	2.54	33	1.5	4.5	4.08	0.30	1	5	5.08	15.20	13.60	1 441
44	Dry.	"	1	1	21.0	$1\frac{1}{2}$	$1\frac{1}{2}$	2.58	40	1.8	4.5	4.17	1.06	1	1	1.72	4.30	3.93	3 966
45	"	"	1	2	21.6	1	2	2.54	40	1.8	4.5	4.17	0.71	1	2	2.54	6.35	5.87	3 404
46	"	"	1	3	20.8	$\frac{2}{3}$	2	2.17	40	1.8	4.5	4.17	0.55	1	3	3.26	8.18	7.59	2 179
47	"	"	1	4	20.8	$\frac{1}{2}$	2	2.12	40	1.8	4.5	4.17	0.42	1	4	4.24	10.71	9.92	1 671
48	"	"	1	5	20.7	$\frac{1}{2}$	$2\frac{1}{2}$	2.21	40	1.8	4.5	.....	0.40	1	5	4.42	11.25	.....	1 559
49	Plastic.	"	1	1	22.6	$1\frac{1}{2}$	$1\frac{1}{2}$	2.50	40	1.8	4.5	4.17	1.08	1	1	1.67	4.12	3.86	4 123
50	"	"	1	2	21.8	1	2	2.54	40	1.8	4.5	4.17	0.70	1	2	2.54	6.43	5.96	2 960
51	"	"	1	3	21.6	$\frac{2}{3}$	2	2.21	40	1.8	4.5	4.17	0.54	1	3	3.32	8.29	7.68	2 027
52	"	"	1	4	21.4	$\frac{1}{2}$	2	2.17	40	1.8	4.5	4.17	0.41	1	4	4.34	10.97	10.17	1 750
53	"	"	1	5	.....	$\frac{1}{2}$	$2\frac{1}{2}$	2.50	40	1.8	4.5	4.17	0.36	1	5	5.00	12.50	11.56	1 465
54	Excess.	"	1	1	22.9	$1\frac{1}{2}$	$1\frac{1}{2}$	2.62	40	1.8	4.5	4.17	1.03	1	1	1.75	4.37	4.05	3 256
55	"	"	1	2	22.9	1	2	2.58	40	1.8	4.5	4.17	0.70	1	2	2.58	6.43	5.96	3 168
56	"	"	1	3	22.2	$\frac{2}{3}$	2	2.25	40	1.8	4.5	4.17	0.54	1	3	3.38	8.34	7.75	2 016
57	"	"	1	4	22.0	$\frac{1}{2}$	2	2.17	40	1.8	4.5	4.17	0.41	1	4	4.34	10.97	10.17	1 670
58	"	"	1	5	.....	$\frac{1}{2}$	$2\frac{1}{2}$	2.62	40	1.8	4.5	.....	0.35	1	5	5.24	12.90	.....	1 400
59	Dry.	Ironclad.	1	2	20.0	$1\frac{1}{2}$	$1\frac{1}{2}$	1.98	33	1.5	4.5	4.08	0.56	1	2	2.64	8.04	7.28	2 777
60	"	"	1	3	20.8	$\frac{2}{3}$	2	2.17	33	1.5	4.5	4.17	0.46	1	3	3.26	9.78	9.06	1 501
61	"	"	1	4	21.2	$\frac{1}{2}$	2	2.12	33	1.5	4.5	4.17	0.35	1	4	4.24	12.85	11.90	1 412
62	Plastic.	"	1	2	22.0	$1\frac{1}{2}$	$1\frac{1}{2}$	1.92	33	1.5	4.5	4.08	0.58	1	2	2.56	7.76	7.04	2 552
63	"	"	1	3	22.7	$\frac{2}{3}$	2	2.21	33	1.5	4.5	4.08	0.45	1	3	3.31	10.00	9.06	1 750
64	"	"	1	4	22.4	$\frac{1}{2}$	2	2.12	33	1.5	4.5	4.08	0.35	1	4	4.24	12.85	11.62	1 434
65	Excess.	"	1	2	24.0	$1\frac{1}{2}$	2	1.80	33	1.5	4.5	4.08	0.62	1	2	2.40	7.25	6.58	2 436
66	"	"	1	3	.....	$\frac{2}{3}$	2	2.29	33	1.5	4.5	4.08	0.44	1	3	3.44	10.23	9.26	1 718
67	"	"	1	4	23.7	$\frac{1}{2}$	2	2.50	33	1.5	4.5	4.17	0.30	1	4	5.00	15.00	13.90	1 594
68	Dry.	Ironclad.	1	2	20.4	1	2	2.50	40	1.8	4.5	4.17	0.72	1	2	2.50	6.25	5.78	3 217
69	"	"	1	3	21.5	$\frac{2}{3}$	2	2.21	40	1.8	4.5	4.17	0.54	1	3	3.32	8.32	7.73	1 929

TABLE No. 2.—Concrete Blocks with Mortar 33% of the Aggregate in Comparison with Blocks in which the Mortar was 40% of the Aggregate.

Serial number.	Brand of cement.	Mortar.	Stone.	Concrete.	Percentage of shrinkage.	Mean ultimate strength of each series, in pounds per square inch.
(1)	(3)	(17)	(18)	(19)	(A)	(20)
		Dry 1 : 1 33% Mortar.				
7	Genesee	1.58	4.71			4 130
29	Wayland	1.50	4.50	4.08		4 267
95	Champion	1.64	5.00	4.53		2 659
	Means	1.57	4.74	4.30	9.3	
		Plastic 1 : 1 33% Mortar.				
12	Genesee	1.75	5.25	4.77		3 505
24	Wayland	2.17	6.50	5.90		4 072
98	Champion	1.58	4.79	4.35		2 593
	Means	1.83	5.51	5.01	9.1	
		Excess 1 : 1 33% Mortar.				
16	Genesee	1.66	4.98	4.54		3 622
39	Wayland	1.75	5.23	4.75		3 764
101	Champion	1.70	5.11	4.64		2 279
	Means	1.70	5.11	4.64	9.2	
		Dry 1 : 1 40% Mortar.				
44	Wayland	1.72	4.30	3.93		3 966
104	Champion	1.57	3.90	3.70		3 125
	Means		4.10	3.82	6.8	
		Plastic 1 : 1 40% Mortar.				
49	Wayland	1.67	4.12	3.86		4 123
107	Champion	1.66	4.16	3.77		2 603
	Means	1.66	4.14	3.82	7.7	
		Excess 1 : 1 40% Mortar.				
25	Genesee	1.67	4.16	3.85		3 191
54	Wayland	1.75	4.37	4.05		3 256
110	Champion	1.68	4.20	4.00		2 196
	Means	1.70	4.24	3.97	6.4	
		Dry 1 : 2 33% Mortar.				
1	Genesee	2.40	7.14	6.90		2 852
30	Wayland	2.19	6.62	6.12		2 888
59	Ironclad	2.64	8.04	7.28		2 777
77	Empire	2.49	7.50	6.80		3 265
96	Champion	2.37	7.14	6.61		2 289
	Means	2.42	7.29	6.74	7.5	

TABLE No. 2—(*Continued*).

(1)	(3)	(17)	(18)	(19)	(A)	(20)
		PLASTIC 1 : 2 33% MORTAR.				
35.....	Wayland.....	2.44	7.39	6.75	.....	2 777
62.....	Ironclad.....	2.56	7.76	7.04	.....	2 552
80.....	Empire.....	2.39	6.71	6.09	.....	2 529
99.....	Champion.....	2.42	7.25	6.58	.....	2 146
.....	Means.....	2.45	7.28	6.62	9.1	.....
		EXCESS 1 : 2 33% MORTAR.				
17.....	Genesee.....	2.21	6.62	6.06	.....	2 524
40.....	Wayland.....	2.04	6.23	5.55	.....	2 847
65.....	Ironclad.....	2.40	7.25	6.58	.....	2 436
83.....	Empire.....	2.63	7.50	6.80	.....	2 899
102.....	Champion.....	2.48	7.50	6.80	.....	1 862
.....	Means.....	2.35	7.02	6.36	9.4	.....
		DRY 1 : 2 40% MORTAR.				
19.....	Genesee.....	2.58	6.43	6.54	.....	2 988
45.....	Wayland.....	2.54	6.35	5.87	.....	3 404
68.....	Ironclad.....	2.50	6.25	5.78	.....	3 217
86.....	Empire.....	2.52	6.33	5.87	.....	3 413
105.....	Champion.....	2.06	5.18	4.90	.....	2 204
.....	Means.....	2.44	6.11	5.79	5.3	.....
		PLASTIC 1 : 2 40% MORTAR.				
22.....	Genesee.....	2.25	5.62	5.26	.....	2 962
50.....	Wayland.....	2.54	6.43	5.96	.....	2 960
71.....	Ironclad.....	2.69	6.71	6.22	.....	2 994
89.....	Empire.....	2.56	6.43	5.96	.....	3 101
108.....	Champion.....	2.44	6.08	5.62	.....	2 037
113.....	Buffalo Portland..	2.54	6.43	5.96	.....	1 717
.....	Means.....	2.50	6.28	5.83	7.1	.....
		EXCESS 1 : 2 40% MORTAR.				
26.....	Genesee.....	2.75	6.82	6.19	.....	2 544
55.....	Wayland.....	2.58	6.43	5.96	.....	3 168
74.....	Ironclad.....	2.62	6.51	6.03	.....	2 866
92.....	Empire.....	2.56	6.43	5.96	.....	2 729
111.....	Champion.....	2.48	6.18	5.71	.....	1 711
.....	Means.....	2.60	6.47	5.97	7.1	.....
		DRY 1 : 3 33% MORTAR.				
8.....	Genesee.....	2.97	8.93	8.63	.....	3 065
31.....	Wayland.....	3.12	9.38	8.69	.....	2 056
60.....	Ironclad.....	3.26	9.78	9.06	.....	1 501
78.....	Empire.....	3.25	9.78	8.62	.....	1 992
97.....	Champion.....	3.17	9.59	8.90	.....	1 401
.....	Means.....	3.15	9.49	8.78	7.5	.....
		PLASTIC 1 : 3 33% MORTAR.				
13.....	Genesee.....	3.38	10.14	8.70	.....	2 311
36.....	Wayland.....	3.18	9.57	8.62	.....	2 207
63.....	Ironclad.....	3.31	10.00	9.06	.....	1 750
81.....	Empire.....	3.44	10.23	9.27	.....	2 014
100.....	Champion.....	3.26	9.78	8.87	.....	1 337
2.....	Genesee.....	3.25	9.80	8.82	.....	2 616
.....	Means.....	3.30	9.92	8.89	10.4	.....

TABLE No. 2—(*Continued*).

(1)	(3)	(17)	(18)	(19)	(A)	(20)
		Excess 1 : 3 33% Mortar.				
18.....	Genesee	3.24	9.78	8.96	.....	2 255
41.....	Wayland.....	3.12	9.37	8.50	.....	1 723
66.....	Ironclad.....	3.44	10.23	9.26	.....	1 718
84.....	Empire	3.44	10.23	9.26	.....	1 898
103.....	Champion.....	3.00	9.00	8.16	.....	1 168
.....	Means.....	3.25	9.72	8.83	9.2	.....
		Dry 1 : 3 40% Mortar.				
20.....	Genesee	3.00	7.50	7.22	.....	2 213
46.....	Wayland	3.26	8.18	7.59	.....	2 179
69.....	Ironclad.....	3.32	8.32	7.73	.....	1 929
87.....	Empire	3.23	8.03	7.43	.....	2 376
106.....	Champion	3.20	8.03	7.44	.....	1 400
5.....	Genesee.....	3.25	8.12	6.78	.....	2 423
.....	Means.....	3.21	8.03	7.36	8.4	.....
		Plastic 1 : 3 40% Mortar.				
23.....	Genesee.....	3.26	8.03	7.44	.....	2 054
51.....	Wayland.....	3.32	8.29	7.68	.....	2 027
72.....	Ironclad.....	3.38	8.38	7.70	.....	1 839
90.....	Empire.....	3.32	8.33	7.75	.....	2 023
109.....	Champion.....	3.29	8.18	7.58	.....	1 193
114.....	Buffalo Portland..	3.29	8.18	7.58	.....	958
.....	Means	3.31	8.23	7.62	7.3	.....
		Excess 1 : 3 40% Mortar.				
27.....	Genesee.....	3.50	8.80	8.00	.....	2 107
56.....	Wayland.....	3.38	8.34	7.75	.....	2 016
75.....	Ironclad.....	3.40	8.50	7.86	.....	2 083
93.....	Empire.....	3.44	8.58	8.00	.....	1 913
112.....	Champion,.....	3.44	8.65	.....	.....	1 127
.....	Means	3.43	8.57	7.90	7.8	.....
		Dry 1 : 4 33% Mortar.				
9.....	Genesee.....	4.00	12.19	11.79	.....	2 059
32.....	Wayland	4.24	12.86	11.66	.....	1 810
61.....	Ironclad.....	4.24	12.85	11.90	.....	1 412
79.....	Empire.....	4.26	12.85	11.65	....''''....	1 666
.....	Means	4.18	12.69	11.75	7.4	.....
		Plastic 1 : 4 33% Mortar.				
14.....	Genesee	4.28	12.86	11.70	.....	2 036
37.....	Wayland.....	4.24	12.80	11.34	.....	1 600
64.....	Ironclad.....	4.24	12.85	11.62	.....	1 434
82.....	Empire	4.38	13.24	12.00	.....	1 523
.....	Means.....	4.28	12.94	11.66	9.9	.....
		Excess 1 : 4 33% Mortar.				
3.....	Genesee...	3.83	11.62	9.54	.....	2 416
42.....	Wayland.....	4.24	12.71	11.70	.....	1 767
67.....	Ironclad	5.00	15.00	13.90	.....	1 594
85.....	Empire.....	4.42	13.24	12.00	.....	1 488
.....	Means	4.37	13.14	11.78	10.0	.....

TABLE No. 2.—(*Concluded*).

(1)	(3)	(17)	(18)	(19)	(A)	(20)
		Dry 1 : 4 40% Mortar.				
21.....	Genesee.....	4.34	10.97	.....	.....	1 972
47.....	Wayland.....	4.24	10.71	9.92	.....	1 671
70.....	Ironclad	4.12	10.46	9.70	.....	1 652
88.....	Empire.....	4.24	10.71	9.91	.....	1 806
.....	Means	4.24	10.71	9.84	8.1	.....
		Plastic 1 : 4 40% Mortar.				
24.....	Genesee.....	4.34	10.90	10.16	.....	1 828
52.....	Wayland.....	4.34	10.97	10.17	.....	1 750
73.....	Ironclad	4.46	11.25	.....	.....	1 544
91.....	Empire.....	4.26	10.71	9.93	.....	1 726
.....	Means	4.35	10.96	10.09	8.0	.....
		Excess 1 : 4 40% Mortar.				
6.....	Genesee.....	4.25	10.63	8.31	.....	2 132
28.....	Genesee.....	4.16	10.40	.....	.....	1 846
57.....	Wayland.....	4.34	10.97	10.17	.....	1 670
76.....	Ironclad	4.54	11.25	10.43	.....	1 508
94.....	Empire.....	4.34	10.97	.....	.....	1 472
.....	Means	4.33	10.84	9.64	11.0	.....
		Dry 1 : 5 33% Mortar.				
10.....	Genesee.....	4.92	14.70	14.20	.....	1 664
33.....	Wayland.....	5.16	15.41	14.39	.....	1 537
.....	Means	5.04	15.05	14.29	5.1	.....
		Plastic 1 : 5 33% Mortar.				
15.....	Genesee.....	5.00	15.00	13.73	.....	1 801
38.....	Wayland.....	5.00	15.00	13.60	.....	1 586
.....	Means	5.00	15.00	13.66	8.3	.....

TABLE No. 3.

Summary of Concrete Tests in Table No. 2.

	33%				40%			
	Number of Blocks.	Col. (17) of Table No. (2).	Col. (18) of Table No. (2).	Col. (19) of Table No. (2).	Per cent. of shrinkage.	Number of Blocks.	Col. (17) of Table No. (2).	Col. (18) of Table No. (2).	Col. (19) of Table No. (2).	Per cent. of shrinkage.
1 to 1 Mortar.										
Dry	3	1.57	4.74	4.30	9.3	2	1.64	4.10	3.82	6.8
Plastic	3	1.83	5.51	5.01	9.1	2	1.66	4.14	3.82	7.7
Excess	3	1.70	5.11	4.64	9.2	3	1.70	4.24	3.97	6.4
2 to 1 Mortar.										
Dry	5	2.42	7.29	6.74	7.4	6	2.44	6.12	5.89	3.8
Plastic	4	2.45	7.28	6.62	9.1	6	2.50	6.28	5.83	7.2
Excess	5	2.35	7.02	6.36	9.4	5	2.60	6.47	5.97	7.7
3 to 1 Mortar.										
Dry	5	3.15	9.49	8.78	7.5	6	3.21	8.03	7.36	8.4
Plastic	6	3.30	9.92	8.89	10.4	6	3.31	8.23	7.62	7.4
Excess	5	3.25	9.72	8.83	9.2	5	3.43	8.57	7.90	7.8
4 to 1 Mortar.										
Dry	4	4.18	12.69	11.75	7.4	4	4.24	10.71	9.84	8.1
Plastic	4	4.28	12.94	11.66	9.0	4	4.35	10.96	10.09	7.9
Excess	4	4.37	13.14	11.78	10.4	5	4.33	10.84	9.64	11.1
5 to 1 Mortar.										
Dry	2	5.04	15.05	14.29	5.1	1	4.42	11.25	.....	.....
Plastic	2	5.00	15.00	13.66	9.1	1	5.00	12.50	11.56	7.5
Excess	1	5.08	15.20	13.60	10.5	1	5.24	12.90	.....	.....

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

STREET GRADES AND CROSS-SECTIONS IN ASPHALT AND CEMENT.

Discussion.*

By Messrs. J. M. EVANS and A. F. HARLEY.

J. M. EVANS, Jun. Am. Soc. C. E.—Judging from the speaker's experience in street work in New York City, the double curve would find little or no place of usefulness there; the objections already mentioned in the discussion being sufficient to exclude it. One objection, however, which has not been mentioned and yet is of grave importance, is the impossibility of keeping such a step in a sanitary condition, as it would afford a convenient repository for filth. In New York City it is the custom to place hydrants, lamp-posts, fire-alarm posts, etc., very near the curb. In the double-curb arrangement these would naturally be set in the tread of the step, and the space around them would soon become filthy and unsightly. It would be more difficult to keep the step clean than the gutter, as no large flow would ever reach the step to cleanse it, while flushing and rains would thoroughly cleanse the gutter. Mr. Evans.

The methods outlined by Mr. Tillson are much more advantageous than those described by the author, and this is particularly true of the method of making the step at the stoop or area line. It is a common sight to see a house of the low-basement style where there are four or five steps leading to the basement floor, and from eight to twelve leading up to the first floor. When these houses are located on

*Continued from March, 1899, *Proceedings*. See February, 1899, *Proceedings* for Paper, by Robert P. Woods, Jun. Am. Soc. C. E., on this subject.

Mr. Evans. the higher side of the street, the speaker can see no reason why the sidewalk could not be depressed from 5 to 8 ins., thereby doing away with one or two of the steps down to the basement floor and adding them to the steps to the first floor. This increase in the number of steps to the first floor might be considered an objection, but hardly a serious one, as in the same block the number of steps often varies from three or four to a dozen. It would seem that it could, therefore, be easily adjusted. In business blocks, where there is a longitudinal grade, it is the custom to provide a series of steps in the coping of the area wall which vary from nothing at the upper end to 6 or even 8 ins. at the lower end. It seems that there ought to be no serious objection to using practically the same method on transverse grades, providing the whole street can be improved thereby, and it is manifest that many streets could be improved in this manner.

Numerous petitions are made by residents for improved pavements, yet, so far as the speaker knows, very few of the petitioners are willing to submit to even the slightest inconvenience for the sake of improvement, much less would they be likely to submit to any lowering of the sidewalk, or anything that would touch their pocket-books. The speaker knows of instances where it was even impossible without process of law to get owners to make the slight repairs to their sidewalk made necessary by setting a new curb. It would seem, therefore, that some form of legislation would be necessary in order to provide for these adjustments, in repaving a street.

The speaker recalls an instance where a catch-basin was left stranded on the summit of a pyramid by changing the cross-section of the street at that intersection. This was caused by a difference of some 2 or 3 ft. in the curb levels. The cross-section of the new pavement was the already mentioned "straight line with 1-in. crown." What the old cross-section was, the speaker does not know, but he remembers that from 10 to 20 ins. of material was removed from the old crown, in reconstruction. This left the catch-basin in its exalted position which was determined by the unchangeable law of the sidewalks. Had the authorities been able to carry the improvement to the property line, a step at the area line would have solved the difficulty, improved the street, and kept the drainage from flooding diagonally across the street, deluging the cross-walks and overflowing the shallow gutters on the lower side of the street.

An interesting question is presented by the method of taking care of street drainage mentioned by Mr. Owen, in cases where under-trolleys are operated, for the reason that much, if not all, of the water will flow into the slot and, doubtless, cause much trouble and annoyance in operation. As to whether a city can so construct its streets as to make a storm sewer out of an under-trolley conduit is probably a question for the courts. Granted that it has such a right, it seems

hardly fair to citizens and patrons of the road, to say nothing of the company, that they should be subjected to the inconvenience and irritation of blockades incident to "short circuits," "burn-outs," etc., caused by improper city drainage. In New York City, where one, from necessity rather than choice, must live far from business, and depend upon street railways for transportation, the latter should be considered as a necessary adjunct to business rather than a necessary evil. Mr. Evans.

A. F. HARLEY, M. Am. Soc. C. E.—The problem of street grades and cross-sections is, perhaps, one of the first that confronts a city engineer, and is one in which, between his own judgment and the wishes of the various property owners, he is likely to find trouble. Mr. Harley.

In considering the subject, great difference should be made in the treatment of business and residence streets. Good renting property on business streets should be provided with such sidewalk grades in front of the various stores as to cause the pedestrian to literally roll into the store he may be passing. No such thing as a step should ever be provided near the building line, but if the grade of the sidewalk is not flush with the floor level, the entrance to the store should be "bayed," and a cast-iron foot-plate, connecting the two elevations, should be set in. On business streets it would not be objectionable, if it were found absolutely necessary, to maintain a sidewalk grade even 18 ins. above the gutter. We know that when this is the case, the street, or rather the original surface of the ground, has invariably a hill-side cross-section, and at each end of the block the grade at intersecting streets would make a tread from the sidewalk to the gutter of not over 6 or 8 ins.

It is the writer's observation that store-keepers prefer this high curb, as it requires pedestrians, ladies especially, to promenade the full length of the block and pass other stores, while intending to visit only one in the row. Then, again, the facility for unloading, already mentioned by Mr. Owen, is a point well made.

A municipal engineer should always remember that the great majority of the citizens walking over a street surface do not carry in their heads an engineer's eye, and, so long as the water does not remain on the surface, they could not tell which way the water flowed after the rain had ceased, and especially could they not detect a hill-side cross-section, *i. e.*, one in which the summit has been shifted from the center toward the higher side, nor a warped surface that runs out in 100 to 200 ft., spoken of by Mr. North. Differences of elevation on the surface of the street paving proper, as a rule, are not detected, even when the water in the gutters on each side of the street runs in opposite directions for a short distance, as might be the case at the intersection of streets that had grades of say, 4 ins. and 4 ft., respectively.

The writer holds, with Mr. Tillson, that a street should always have some crown, to allow of slight wear to the surface, which is

r. Harley. always most in the center, provided there is no electric railroad on the street. But on the other hand, on narrow business streets, say, 30 ft. in width, occupied for more than half their width in the center by two electric street railroad tracks, in the writer's opinion, it is an open question whether it would not be desirable to throw all the water to the center, provided it could be collected and carried off, as it would be the only moving thing that would not meet its death by being run over in crossing the track; while the surface of the street close to the curb line would be more available for vehicular travel; in fact, this gutter space would be gained. In a street railroad, especially one in which the use of the ordinary T-rail is permitted, the paving between the rails is given a crown of, say, 1 in.; thus little streamlets are formed and are compelled to run the entire length of the longitudinal grade on the street, which is bad for both the railroad and the pavement, as it is almost impossible to make water-tight joints between the rails and the adjoining pavement. By the adoption of a central gutter the necessity for making these joints absolutely water-proof is avoided. Were it not for the appearance the writer would like to see the paving between the rails concaved, instead of convexed, and cast-iron gratings having the same concavity placed over catch-basins built in the center of the track at the required intervals.

The writer does not think it would be very much to ask of street railroads using the underground electric system, to make provision in their conduits to carry off the surface water that might be led into them in this manner.

Another point is, that while a very large majority of people, in walking or driving over a street, do not notice the grades on a continuous asphalt surface, as already stated, they readily detect the difference in the height of the curbs on the street over which they are driving; even such a small difference as 4 is to 6 ins. It would, therefore, be advisable on all residence streets to make the height of the exposed curbing the same on both sides of the street, even though the surface of the pavement may have a "hill-side cross-section" or be a "warped surface."

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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GENERAL CRITERION FOR POSITION OF LOADS CAUSING MAXIMUM STRESS IN ANY MEMBER OF A BRIDGE TRUSS.

Discussion.*

By F. H. CILLEY, Esq.

Mr. Cilley.

F. H. CILLEY, Esq.—The author in his exposition of criteria for the position of loads causing maximum stress offers an interesting extension of the well-known rule for loading for maximum moment at a given section of a beam, that the average load on both sides of the section shall be equal. His principle applies to all cases in which the influence line consists of three lines, which include most cases arising in connection with statically determined frameworks. And in the case of an influence line whose end lines intersect in the region of the middle line the rule still possesses a simple objective interpretation like that of the earlier moment rule, which applied only to two-line influence lines.

It may, perhaps, be of interest to note some very simple and wholly general facts in connection with influence lines. First, the influence line for the stress in any bar of any framework whatsoever, determinate or indeterminate, consists simply of a series of straight lines, if we neglect secondary stresses. It follows, therefore, that the variation in stress in any such bar, due to the movement along the load line of a

*This discussion (of the Paper by L. M. Hoskins, Esq., printed in *Proceedings* for March, 1899), is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to May 26th, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

Mr. Cilley. series of concentrated loads maintained at invariable distances apart, is a linear function of the amount of movement of the load. A linear function of a variable can become a maximum or minimum only at a point of discontinuity of the function, that is to say, in this case, only when some load passes an angle of the influence line. For a maximum it is evident that the angle must be concave toward the zero line in a region of positive influence, convex toward the zero line in a region of negative influence; and for a minimum, the reverse is the case.

A particularly objective and interesting general rule as to the position of loads for maximum influence is found in the following:

Imagine (in Fig. 9), a track of the form of the influence line with positive part down and zero line horizontal. Imagine a series of small

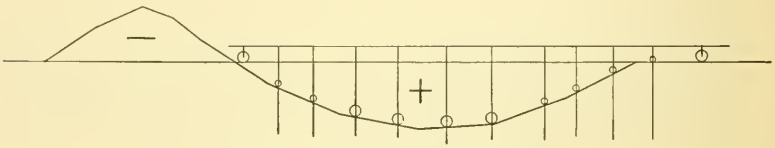

FIG. 9.

wheels, the weights of which are proportioned as those of an actual series of loads, and which are free to move vertically, independently; but horizontally only all together, that is, with constant horizontal spacing. A position of stable equilibrium of this series of wheels on the influence line track will correspond to a position for maximum influence of the corresponding actual loads with similar spacing. Here it is very evident that some load must be at an angle concave from above. The reasons for this rule will readily be perceived. Its immediate source, if the writer's memory is not in error, is an Italian work on graphic statics.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will hereafter be reproduced in the Volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

DAVID LEONARD BARNES, M. Am. Soc. C. E.*

DIED DECEMBER 15TH, 1896.

David Leonard Barnes was born in Smithfield, R. I., August 23d, 1858. When he was but eleven years old his father died, and the young man was led thus early in life, to the exercise, in a serious way, of self-reliance and responsible judgment. At the age of fifteen he entered a civil engineer's office in Providence, where his work included town surveys and the laying out of streets. His mind, however, was set upon obtaining an education better than that afforded by the routine of a surveyor's office, and he spent the three years following 1876 in Brown University and the Massachusetts Institute of Technology as a special student, while the vacations were occupied in engineering work. He received the degree of Master of Arts from Brown University, but took no degree from the Institute of Technology. Although surveying and municipal work were his earliest engineering employments, both the bent of his mind and his special studies inclined to the mechanical side of civil engineering, and it was in this that his reputation was won. He served three years following 1879 in the machine shops and drafting rooms of the Rhode Island Locomotive Works, at Providence, R. I., and the Hinckley Locomotive Works, Boston, Mass. He was Chief Draftsman at the Rhode Island Works when, in 1884, he left to become the Mechanical Engineer and Chief Draftsman of the New York Locomotive Works at Rome, N. Y., which were built in that year. Returning to Providence he passed three more years as Chief Draftsman and Mechanical Engineer of the locomotive works with which he had begun his mechanical practice.

Mr. Barnes had now become well known as a designer of railway rolling-stock and an authority on many other matters connected with railway operation, and in 1888 he entered upon consulting practice, with offices in Chicago, Ill., and New York City. During his service in the locomotive shops he had designed and superintended the construction of about 800 locomotives, and also a large number of stationary and steamboat boilers and special tools. At the Rhode Island

* Memoir prepared by the Secretary from papers on file and from various sources.

works he had planned and supervised the construction of dynamos and electric motors for street railway equipment, and designed the shops of the Boston and Maine Railroad. The shops of the Bentley-Knight Electric Railroad Company were completed, fitted up, equipped and put in operation under his direction. The equipment of the Allegheny, Pa., Electric Street Railway was also designed under his supervision. At the Rome works, Mr. Barnes designed, purchased and superintended the construction of a large part of the machinery installed, as well as of the first locomotives built at the works. Other work of a general character included the design, for the Providence Ship Windlass Company, of steam windlass and hoisting gear for the United States Government. In 1888 he became consulting engineer to the *Railroad Gazette*, New York City, a relation which was later broadened to engagement as mechanical editor, which connection was maintained until his death.

Mr. Barnes' work as a consulting engineer included designs, tests, inspection and reports covering the whole range of railway rolling-stock and operating devices, manufacturing plants, iron-work for buildings and steam-engine construction. He was engaged as consulting engineer by the London Central Subway Railway and by the New York Underground Railroad. The Chicago and South Side Rapid Transit Railroad, known as the Alley Elevated, retained him to have general supervision of the design and construction of its rolling equipment, signaling, lighting and shop plant. He made a radical departure from the current practice on elevated railways, by introducing, in 1893, compound locomotives on the Chicago line. When the Baldwin Locomotive Works and the Westinghouse Electric and Manufacturing Company decided to join forces in an endeavor to realize the union of the essential features of the standard steam locomotive and the accepted practice in the arrangement of mechanism for electric traction. Mr. Barnes was engaged to take charge of the work. He entered upon this labor with enthusiasm, and the designs produced have been commended for their robust simplicity and adaptability to the work required.

Mr. Barnes was endowed with unusual strength and energy, both physical and mental. He was, by nature, a mathematician, and had strong powers of analysis and reasoning. He found pleasure in the discussion of technical questions, and his arguments were marked by a directness of address to the main issues, which irrelevant details were never allowed to obscure. He had the scientific love of truth, and his published work was of a nature to stimulate his fellow engineers to thought and effort. A rapid and systematic worker, in addition to his editorial labors for the technical press, he made many contributions to the publications of the Master Car Builders' Association, the Master Mechanics' Association and the Western Railway Club, and his work

included numerous reports on locomotive and car construction, and on electricity as a motive power for railroads. He carried to completion a revised edition of "Compound Locomotives," taking up the work after the death of Arthur Tannatt Woods, the author, and about a year before his death he wrote a book on "Electric Locomotives."

Mr. Barnes became a Member of the American Society of Civil Engineers, July 2d, 1890. He was also a Member of the American Society of Mechanical Engineers, an Associate Member of the Institution of Civil Engineers, a Member of the Engineers' Club of New York and of various societies of railroad operating officers. He was married April 7th, 1896, to Ida, daughter of Colonel B. J. D. Irwin, U. S. A. (Retired), formerly Assistant Surgeon-General. In the summer of 1896 Mr. Barnes relaxed business engagements for a trip to Europe in hope of improved health, but was soon obliged to return for medical treatment. During the summer his strength steadily declined, but he did not cease work until two weeks before his death.

WILLIAM ROBERTS MICHIE, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 2D, 1899.

This young engineer, cut off in his prime, was born at Yanktown, McDowell County, N. C., September 22d, 1866. His father, Peter S. Michie, now Professor of Natural and Experimental Philosophy at the United States Military Academy at West Point, had served with distinction through the Civil War, had risen to the rank of General of Volunteers, and was at that time in the regular army. As the engineer officer of the Army of the James, Gen. Michie had rendered distinguished services, so that the son came by his taste for engineering in a manner almost hereditary. It was the desire of the parents that the young man should be educated at the United States Military Academy, but, owing to a slight trouble with his eyes, he was sent instead to Dartmouth College, where he was graduated as a civil engineer, in April, 1889, from the Thayer School of Civil Engineering. Professor Robert Fletcher, the head of the Thayer school at that time, was a graduate of West Point, so that, although Mr. Michie was denied admission to the Military Academy on account of a disability which was never perceptible, he nevertheless obtained an education modeled very much on the West Point lines. He showed his ability at once on graduation to such an extent that after a brief service on the St. Louis and San Francisco Railway he rose to the position of assistant engineer. He had charge, for that railway, of the construction of a pier and one span of the Red River Bridge, and made surveys for wing dams to change the current of the Red River.

He entered the service of the Pennsylvania Railroad Company, April 20th, 1889, upon the recommendation of Mr. Frank Thomson, and had occupied the following positions on this celebrated railroad: 1889, chainman on construction of the Ardara Bridge, also at Viaduct and South Fork Bridges until August, when he became rodman on the construction of the Wall repair shops; 1890, in July, Acting Assistant Engineer in the construction of a new yard at Altoona; in October promoted to Assistant Engineer, Altoona yards, surveys for South Fork Railroad and part of its construction; construction of Wall yard and round houses, a large railroad yard near Pittsburg; construction of the Sang Hollow extension, approximately fifteen miles; 1895, construction of third and fourth track work on the Pittsburg Division and at various points on the Johnstown Branch; 1896, completing the Wilmerding Branch, and on the new line at Nineveh and the Radebaugh Tunnel; 1897, construction work on the Pittsburg Division, change of line and additional tracks at various points; 1898, construction work, additional track-work in Johnstown and vicinity and the Portage Tunnel; 1899, resigned January 10th.

* Memoir prepared by William H. Wiley, M. Am. Soc. C. E.

To show what rare capacity he exhibited at the beginning of his career some details are given: In November, 1890, within six weeks' time, he made the surveys for the extension from South Fork to Dunlo, and the line was afterward built on his location. The change of line from Portage to Tilly was also constructed practically in accordance with Mr. Michie's surveys, made in 1891. In that year, also, Mr. Michie had charge of much important work on the Pittsburg Division. Entering upon the construction of the extensive Wall yards, in the early stages of the work, he carried it to completion in December, 1892, and built at the same time a 44-stall roundhouse. Under his direction the line between Turtle Creek and Brinton was changed, and the Wilmerding Branch Railroad was constructed.

After the Johnstown flood Mr. Michie was engaged in the reconstruction of the track and was inspector in charge of the construction of the celebrated Viaduct Bridge near South Fork, Pa. This structure, which is notable as one of the largest arch bridges in the United States, has two 80-ft. spans and is over 100 ft. in height. He also had charge of the extensions to the large arch bridges over the Juniata River; five spans of 60 ft. each. In addition to the record of service given as above by years, Mr. Michie was in charge of numerous important surveys for changes of the line and branches on the Philadelphia and Erie, and on the Northern Central Railroads as well as on the Baltimore and Potomac Railroad, near Washington, D. C. He had full charge of building the Radebaugh Tunnel, 2 100 ft. long, near Greensburg, Pa., and was so successful with this work that he was detailed to superintend the rebuilding of the celebrated Portage Tunnel at Gallitzin, Pa. This was the last construction work he undertook for the railroad company, as he resigned his position in January, 1899, to accept an advantageous appointment in Cuba. Mr. Michie was quite familiar with the uses of steam and with steam engines and machinery generally, and could handle a locomotive, apparently, as well as the engine-runner.

This, in brief form, is the record of an engineer who, had he lived, would have attained great distinction in his profession. The writer knew him when he was a boy, and he possessed such lovable traits of character that he drew everyone toward him. As a man he justified all these early characteristics and was one of the most popular men on the Pennsylvania Railroad. For the preparation of this memoir there were available not only many personal letters from his associates and from those under his immediate charge, but also newspaper extracts containing accounts of Mr. Michie and his work, and all of these expressed the sentiment of a personal loss sustained by the writers.

Mr. Michie was elected a Junior of the American Society of Civil Engineers, February 28th, 1893, and an Associate Member, October 7th, 1896.

HISTORICAL SKETCH

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS,

By Charles Warren Hunt, M. Am. Soc. C. E.

Cloth, 6 x 9 Inches.

Printed by order of the Board of Direction of the American Society of Civil Engineers, to be sold only on subscription. The proceeds to be devoted exclusively to the fund for the New Society House.

At the Annual Meeting, January 19th, 1898, the following facts in regard to the subscription to this book were brought out:

Two thousand copies were printed; 300 were bound in full morocco, of which 216 have been sold at $10 per copy, the resulting net profit being $943.06. Seventeen hundred copies, which have been paid for, are still on hand, and the Board of Direction was requested to consider the propriety of offering to the membership these copies bound in a less expensive style and at a reduced price, the net proceeds to be applied to the building fund.

In compliance with this request it has been decided to bind as many copies as are necessary to supply the demand, in a handsome cloth binding and to supply them at $5 per copy.

This action has been taken in the belief that many members will welcome the opportunity of contributing something to the building fund.

There are a few copies still on hand of the first lot bound and these can be obtained by those who so desire at $10 per copy.

Orders should be sent to the Secretary.

The book begins with a brief statement of the first movement to form a National Society of American Engineers in 1839. The organization of the American Society of Civil Engineers and Architects in 1852 is then described, a list of its promoters and charter members given, and the work accomplished in its first two years of life sketched. The reorganization of the Association in 1867 and the important events in its career from that date to 1873, when the first publication was issued, are then given in chronological order. Succeeding chapters are under the following heads: Locations Occupied by the Society, Library, International Exhibitions, Publications Badge, Constitutional Changes and Work Accomplished. Under the head of "Comparative Growth of National Engineering Societies" short sketches of the Institution of Civil Engineers and the Société des Ingénieurs Civils are given. The illustrations consist of 35 half-tone portraits of past officers of the Society and one diagram, all handsomely printed on heavy paper.

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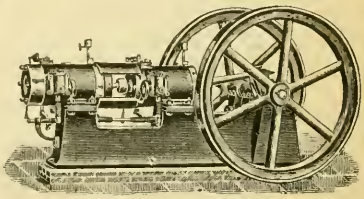

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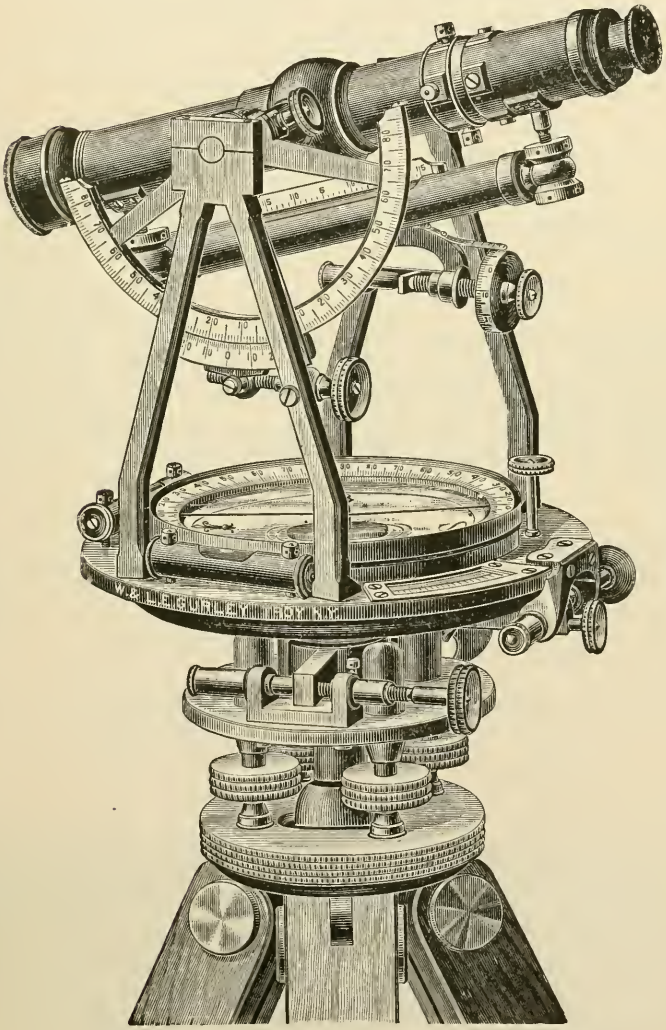

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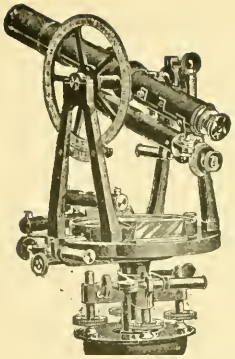

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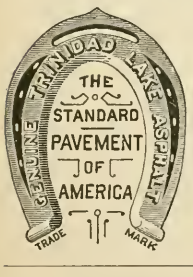

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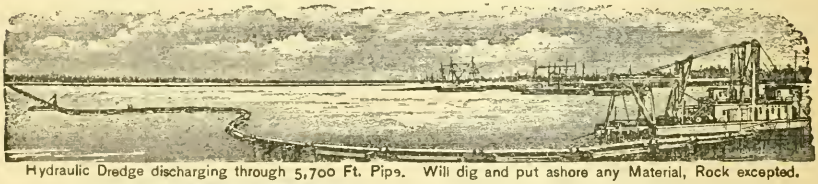

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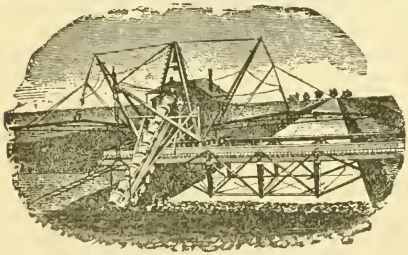

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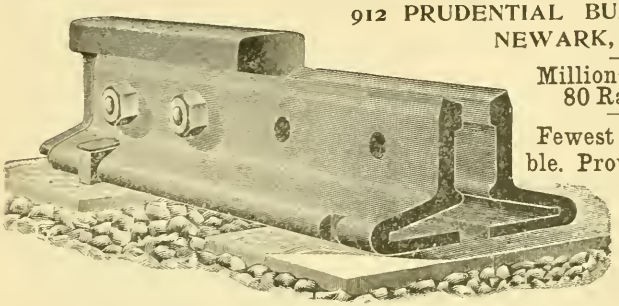

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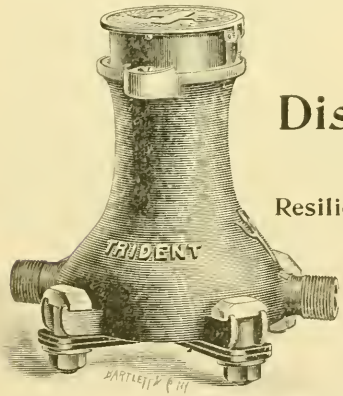

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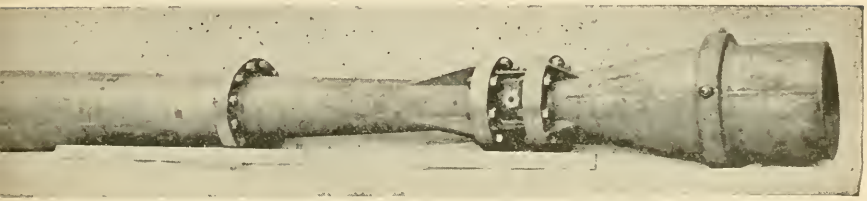

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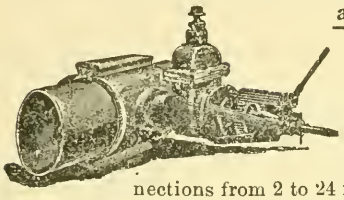

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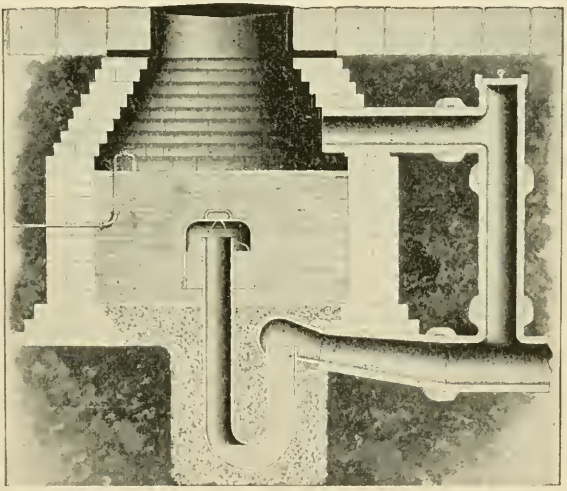

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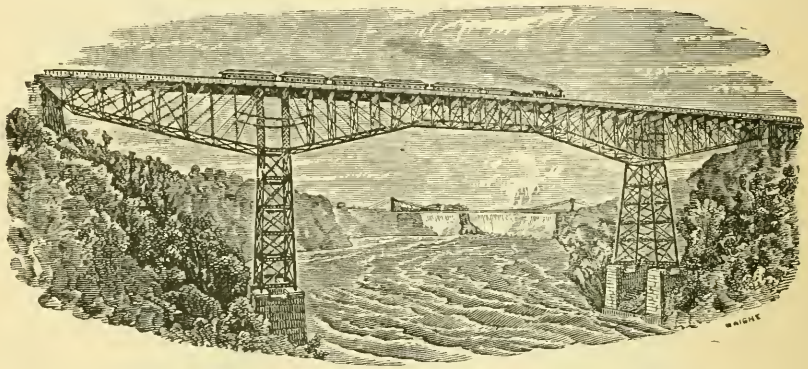

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OF

CIVIL ENGINEERS

May, 1899

PROCEEDINGS = VOL. XXV—No. 5

Published at the House of the Society, 220 West Fifty-seventh Street, New York,
the Fourth Wednesday of each Month, except June and July.

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Vol. XXV. MAY, 1899. No. 5.

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publications.

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American Society of Civil Engineers.

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Special Committees.

ON UNIFORM STANDARD TIME:—Sandford Fleming, Charles Paine, Theodore N. Ely,

ON ANALYSIS OF IRON AND STEEL:—Sub-Committee of the American Society of Civil Engineers (of the International Committee on Standards for the Analysis of Iron and Steel, of which Prof. J. W. Langley is Chairman)—Charles B. Dudley, William Metcalf, Thomas Rodd.

ON UNITS OF MEASUREMENT:—George M. Bond, William M. Black, R. E. McMath, Charles B. Dudley, Alexander C. Humphreys.

ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Noble, George S. Webster, O. M. Carter, W. B. W. Howe, Louis C. Sabin, H. W. York.

The House of the Society is open every day, except Sunday, from 9 to 22 o'clock.

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Vol. XXV. MAY, 1899. No. 5.

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INSTITUTED 1852.

PROCEEDINGS.

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MINUTES OF MEETINGS.

OF THE SOCIETY.

May 3d, 1899.—The meeting was called to order at 20.50 o'clock, Vice-President Edward P. North in the chair; Charles Warren Hunt, Secretary, and present, also, 64 members and 11 visitors.

The minutes of the meetings of April 5th and 19th, 1899, were approved as printed in *Proceedings* for April, 1899.

The Secretary presented a paper by John W. Hill, M. Am. Soc. C. E., entitled "Comparative Tests of Bituminous Steam Coals," and the subject was discussed by Messrs. William Kent and Henry C. Meyer, Jr.

The Secretary announced the following deaths: FRANCIS RINECKER, elected Member August 7th, 1872; died April 10th, 1899. CHARLES HERBERT SWAN, elected Member March 16th, 1870; died April 17th, 1899. ALFRED EPHRAIM HUNT, elected Member September 1st, 1886; died April 26th, 1899.

Announcement was made that the following candidates were elected by the Board of Direction, May 2d, 1899:

As Associate.

William Drumm Sell, Logan, W. Va.

As Juniors.

David Elder Baxter, New York City.
George Harold Bouton, Jersey City, N. J.
Robert Duncan Coombs, Jr., Phœnixville, Pa.
Sumner Gowen, Phœnixville, Pa.
Don Alexander MacCrea, Little Rock, Ark.
Henry Bennett Machen, New York City.
Elbert Clyde Macy, Marshalltown, Iowa.
Ludlow Lawrence Melius, Albany, N. Y.
Charles Sterling Millard, Indianapolis, Ind.
Charles Church More, Ithaca, N. Y.
James William Skelly, St. Louis, Mo.

Ballots were canvassed and the following candidates declared elected:

As Members.

Fred Milton Bisbee, Springfield, Mo.
Franklin Allen Hinds, Watertown, N. Y.
Isaac Wayne von Leer, Havana, Cuba.
Charles Hedges McKinstry, Key West, Fla.
Dabney Herndon Maury, Jr., Peoria, Ill.
Francis Lee Stuart, Washington, D. C.
George Tatnall, Wilmington, Del.

As Associate Members.

Alfred Pancoast Boller, Jr., Honolulu, H. I.
Jaquelin Marshall Braxton, Key West, Fla.
Fred Goodrich Frink, Moscow, Idaho.
Ferdinand Finney Harrington, Philadelphia, Pa.
Edward Fulbister Kenney, Moore, Pa.
Kingsley Leverich Martin, Brooklyn, N. Y.
George Cotner Mason, New York City.
Frederic Vernon Pitney, Morristown, N. J.
William Babcock Poland, Cincinnati, Ohio.
Henry Wray Preston, Athens, Pa.
Charles Stewart Towle, New York City.
Bernard Matthew Wagner, Brooklyn, N. Y.

The Secretary announced that at the meeting of the Board of Direction May 2d, 1899, the ballot on the reconsideration of William Copeland Furber and Charles Henry Miller was canvassed, and Messrs. Furber and Miller were declared elected Members of the Society.

The Secretary announced that arrangements for the Annual Convention are being perfected, and a circular will soon be issued giving information in regard to the matter.

Announcement was made that, owing to the usual mid-month meeting of June occurring only a few days preceding the week during which the Annual Convention will be held, the Board of Direction had decided that no meeting of the Society will be held on June 21st, 1899.

Announcement was made that the Board of Direction had authorized the holding of meetings of the Juniors of the Society on the second Wednesdays of May and June. A committee of the Juniors, consisting of Messrs. J. M. Evans, M. E. Evans and E. D. Knap has been appointed to propose subjects for discussion, and attend to such other work as is necessary in connection with these meetings. Notice of the subjects selected will be issued to all Juniors by the Secretary.

Adjourned.

May 17th, 1899.—The meeting was called to order at 20.45 o'clock, Vice-President Edward P. North in the chair; Charles Warren Hunt, Secretary, and present, also, 61 members and 4 visitors.

A paper by Walter W. Curtis, M. Am. Soc. C. E., entitled "The Artificial Preservation of Railroad Ties by the Use of Zinc Chloride," was read by the Secretary, who also presented a written communication on the subject from Mr. J. I. Boggs. The subject was discussed orally by Messrs. Oscar Lowenson, H. S. Haines and Edward P. North.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

May 2d, 1899.—President FitzGerald in the Chair; Charles Warren Hunt, Secretary, and present, also, Messrs. Bensel, North, Owen, Ricketts, Thomson and Schneider.

Routine business in regard to publications and the work of the library was transacted.

The question of an exhibit, under the auspices of the Society, at the Paris Exposition of 1900, was considered.

Reconsideration ballots were canvassed, and William Copeland Furber and Charles Henry Miller were declared elected Members of the Society.

A progress report from the Committee in charge of arrangements for the Annual Convention was received.

The Secretary reported that a suit of the Society for exemption from taxation had been successful; that the taxes for 1898 had been cancelled of record, and that the exemption would probably hold in future.

The resignation of F. G. Darlington as a Member of the Society was accepted.

Meetings of the Juniors of the Society were authorized to be held on the second Wednesdays of May and June, 1899.

It was decided that no meeting of the Society be held on the third Wednesday of June, 1899.

Applications were considered and other routine business transacted.

One candidate for Associate and eleven for Junior were elected.

Adjourned.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society is open every day, except Sunday, from 9 to 22 o'clock.

MEETINGS.

Wednesday, June 7th, 1890, at 20.30 o'clock, a regular business meeting will be held. Ballots for membership will be canvassed, and a paper by George W. Rafter, M. Am. Soc. C. E., entitled "On the Theory of Concrete," will be presented for discussion. This paper was printed in the April number of *Proceedings*.

In accordance with the resolution of the Board of Direction there will be no meeting of the Society on Wednesday, June 21st, 1899.

JUNIOR MEETINGS.

May 10th, 1899.—A meeting of Juniors was held, J. M. Evans, Jun. Am. Soc. C. E., in the chair, and present also twenty-two Juniors. The subject for the evening was "Surveying, in Connection with City Work," and brought out an interesting discussion.

June 14th, 1899.—There will be a meeting of Juniors of the Society for informal discussion of a subject to be subsequently announced.

ANNUAL CONVENTION.

Time and Place.—The Thirty-first Annual Convention will be held at the Stockton Hotel, Cape May, N. J., June 27th to 30th (inclusive).

Meetings.—A meeting of the Board of Direction will be held during the Convention, as required by the Constitution.

There will be a Business Meeting held during the Convention, at which members of the Nominating Committee will be elected, and the time and place for holding the next Annual Convention will be considered. Reports of Special Committees may also be presented.

At one of the meetings the President will deliver the Annual Address.

On one evening Col. Chas. W. Raymond, M. Am. Soc. C. E., will address the Society on the subject of Breakwater Construction: the address will be illustrated with lantern slides.

Meetings will also be held for the discussion of the following professional papers and subjects:

Papers and Subjects for Discussion.—Under the rule recently adopted by the Board of Direction, discussion will be in order on the

following papers and discussions which have been printed in *Proceedings* since January 1st, 1899:

"The Accuracy and Durability of Water Meters."
By JOHN W. HILL, M. Am. Soc. C. E.

"The Approaches and Transportation Facilities of the Paris Exposition of 1900."
By E. L. CORTHELL, M. Am. Soc. C. E.

"Street Grades and Cross-Sections in Asphalt and Cement."
By ROBERT P. WOODS, Jun. Am. Soc. C. E.

"General Criterion for Position of Loads Causing Maximum Stress in Any Member of a Bridge Truss."
By L. M. HOSKINS, Esq.

"Dry Docks—Stone *vs.* Wood."

"Comparative Tests of Bituminous Steam Coals."
By JOHN W. HILL, M. Am. Soc. C. E.

"The Artificial Preservation of Railroad Ties by the Use of Zinc Chloride."
By WALTER W. CURTIS, M. Am. Soc. C. E.

"On the Theory of Concrete."
By GEORGE W. RAFTER, M. Am. Soc. C. E.

In addition to the above, the following subjects have been selected for informal discussion. Members who are unable to attend the Convention are invited to send written communications on any of the subjects, for presentation to the meetings.

(1) Should the use of the method of Wheel-Concentrations be discontinued in determining the Stresses in Railroad Bridges?

(2) In view of present knowledge of the Effect of Repeated Applications of Load, should Fatigue Formulas be used in Bridge Design?

(3) (*a*) Should Stream Contamination by the Sewage of Cities be absolutely prohibited by law?

(*b*) Should the Purification of the Sewage of Cities be compulsory; and is this feasible for Large Cities?

(*c*) Is Filtration the coming solution of the Pure-Water Question for Cities?

(4) What is the Proper Friction Coefficient for use in the design of Riveted Steel Pipe?

(5) What are the economic conditions under which Electricity may be profitably substituted for Steam in the operation of Branch Railroad Lines, and what are the engineering requirements to be considered in such substitution?

(6) What is the present development of the so-called Telferage System for moving either Freight or Passengers? What are the conditions under which that System is preferable to movement by Rail, and what is its adaptability to still further application in competition with Rail Lines?

Transportation.—Through the courtesy of the Pennsylvania Railroad and the Philadelphia and Reading Railway Companies the following arrangements have been effected:

A special train will leave **Jersey City, on Monday, June 26th, 1899,** at an early hour in the afternoon over the Pennsylvania Railroad to Camden and thence to Cape May, arriving in time for dinner.

For members from the West and South, transportation between Pittsburg and Philadelphia, and between Washington and Philadelphia, will be furnished by the Pennsylvania Railroad.

On Saturday, July 1st, 1899, at 8 A. M., a special train provided by the Philadelphia and Reading Railway will leave Cape May for Philadelphia, where various points of interest will be visited and lunch served. The New York party will return on a special train furnished by the Philadelphia and Reading Railway. This train will leave Philadelphia at 2.30 P. M. and arrive in New York at about 4.30 P. M.

Transportation on the above trains will be free to the membership of the Society and the members of their immediate families, and must be secured by application to the Secretary.

Members from the New England States, and all others who can do so conveniently, should arrange to arrive in New York in time to take the special train.

Those who cannot go *via* New York should purchase tickets to Pittsburg or to Washington, and arrange to meet the special at Camden. Transportation from either of these points to Philadelphia and Cape May must be secured by application to the Secretary.

Between Philadelphia and Cape May and between New York and Philadelphia no transportation will be furnished except on the special trains.

Hotel Rates.—The Stockton Hotel, the exclusive use of which has been secured for the Convention dates, will be opened for the Society on Monday, June 26th. The Stockton has accommodation for 750 guests. The rate has been fixed at $3 per day.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(April 12th to May 10th, 1899.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal.* Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings.* Eng. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal.* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal.* Western Soc. of Eng., Monadnock Block, Chicago, Ill.
(5) *Transactions.* Can. Soc. C. E., Montreal, Que., Can.
(6) *School of Mines Quarterly.* Columbia Univ., New York City, 50c.
(7) *Technology Quarterly.* Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Indicator.* Stevens Institute, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 30c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 35c.
(12) *The Engineer* (London). International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c
(14) *Engineering Record*, New York City, 12c
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City, 35c.
(18) *Railway and Engineering Review*, Chicago, Ill.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England.
(22) *Iron and Coal Trades Review*, London, England.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gaslight Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Electrical Engineer*, New York City, 10c.
(28) *Industries and Iron*, London, England.
(29) *Journal*, Society of Arts, London, England.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des Ecôle Spéciales de Gand*, Brussels, Belgium.
(32) *Memoirs et Compt Rendu des Travaux.* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil.* Paris, France.
(34) *Portefeuille Economique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction.* Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mecanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Transactions*, Am. Inst. Elec. Eng., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal.* Military Service Institution, Governor's Island, New York Harbor, 75c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 10c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift des Vereines Deutscher Ingenieure*, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Duesseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereines*, Vienna, Austria.
(54) *Den Tekniske Forenings Tidsskrift*, Copenhagen, Denmark.
(55) *Ingeniören*, Copenhagen, Denmark.
(56) *Teknisk Tidskrift.* Stockholm, Sweden.
(57) *Teknisk Ugeblad*, Christiana, Norway.
(58) *Proceedings.* Eng. Soc. W. Pa. 410 Penn Ave., Pittsburg, Pa. 50c.
(59) *Transactions.* Mining Institute of Scotland, London and Newcastle-upon-Tyne.
(60) *Bridges and Framed Structures*, 358 Dearborn St., Chicago, Ill., 30c.
(61) *Proceedings*, Western Railway Club. 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.

LIST OF ARTICLES.

Bridge.

Curved Girders. A. Francke. (49) IV-VI.
Existing Experiments for Determining the Neutral Axis in Beams of Stone and Cast Iron. E. Roser. (48) Feb. 25.
The Recovery of Iron from Overstrain. (18) April 15.
Chemical and Physical Constitution of Steel. William R. Webster. (60) April, 1899.
Contribution to the Determination of Maximum Moments for Concentrated Loads on Simple Girders. A. Coulmass. (53) April 14.
Draw Span Stresses—Assumptions Made to Determine Them. Malverd A. Howe, M. Am. Soc. C. E. (60) April, 1899.
The Architecture of Bridges. (60) April, 1899.
The Erection of a Steel Viaduct upon the Highland Railway. H. T. White. (63) Part I., 1899.
Failure of the Embabeh Bridge, Cairo. F. E. Robertson. (63) Part I. 1899.
Failure of the Falsework of the Willis Avenue Bridge, New York. (14) April 15; (15) April 21.
New Bridges over the Rhine at Bonn and Duesseldorf. (48) March 25.
The Bridges across the Rhine, near Bonn and Duesseldorf, Germany. Fritz Mueller von der Werra. (13) April 20.
The Bridge Works' Estimating Department. E. M. Scofield. (60) April, 1899.
Erection of Bridge 69, Pennsylvania Railroad. (14) April 22.
A Highway Bridge Floor of Rock Elm. (13) April 13.
Pneumatic Caissons for Ordinary Foundations. A. W. Jones. (60) April, 1899.
Field Riveting in Elevated Railway Work. (40) April 28; (18) April 29; (15) April 28.
Some Examples of Bridge Engineering in Iowa. (14) May 6.
The New Bridge over the Niagara River. (11) April 28.
Strengthening the Cantilever Bridge at Niagara. (40) May 5.
Measurements for Granite Viaduct. E. I. Cantine. (60) April, 1899.
Three-Hinged Melan Arch Bridge. (55) March 4.
Experiences and Lessons from the Life of a Bridge Engineer. (13) May 4; (14) May 6.
Street Bridges over the Arge at Laugenargen. (48) Jan. 7.
Viaduct near Drum and Neuschloss on the North Bohemian Railroad. H. Rosche. (53) Jan., 27.
Iron Trestle on the Waldhoven-Ganning. O. Weltzer. (53) Feb. 24.
Iron Bridges in Austria. (50) Feb. 1.
Modern Spanish Bridge Engineering. (60) April, 1899.
Japanese Bridge Work. (40) April 28.
A Korean Bridge. (14) April 15.
Étude Expérimentale des Ponts Métalliques. (36) April 25.

Electrical.

Telephones. John Gavey. (29) April 14.
Wireless Telegraphy. Rankin Kennedy. (26) April 14.
Picture Telegraphy. (27) May 6.
Callendar's Electrical Recording Apparatus. (26) April 14.
A New Form of Wehnelt Interrupter. Elihu Thomson. (27) May 6.
On the Dimensioning of Field Magnets. George T. Hanchett. (27) April 29.
Bipolar and Multipolar Construction. Alton D. Adams. (27) April 29.
Effect of Inequality of Voltage on Three-Phase Watt-Meters in Measuring Motor Loads. August J. Bowie. (27) April 22.
A New Generator with Minimum Field Distortion. Robert Lundell. (27) April 22.
On a Direct-Current Dynamo without Brushes. Albert Campbell. (26) April 14.
Alternating Current Motors for Single System. O. Lasche. (48) March 18.
Secondary Distributing Systems for Three-Phase Plants. August J. Bowie, Jr. (27) April 29.
Action of a Synchronous Motor as an Inductance and as a Condenser. A. F. McKissick. (27) April 29.
The Electric Lighting of Leith. (26) April 14.
Selection of Incandescent Lamps. Alton D. Adams. (10) May, 1899.
The American Nernst Lamp. (27) April 29.
Electric Power from Windmills. C. L. Crandall, M. Am. Soc. C. E. (15) May 5.
Hammersmith Electricity Works Extensions. (26) April 21.
Data on Electric Power Generation, Glasgow. (14) April 22.
Lewiston, Idaho, Long Distance Transmission. (27) April 22.
Switchboard of the Chelsea Power Station, Lynn and Boston Railway. H. Wray Weller. (27) April 15.
The Use of Electricity as a Motive Power in Modern Steel Works and Blast Furnaces. (27) April 15.
Storage Batteries and Railway Power Stations. Robert McA. Lloyd. (42) Feb., 1899.
Earth Currents and Electric Traction. (12) April 21.
Electrolysis as an Unsolved Municipal Problem. (27) April 29.
The U. S. Signal Corps and the Philippines' Cable. (27) May 6.
High Efficiency Transformers for England. (27) May 6.
Sur un Interrupteur Electrolytique. A. Wehnelt. (36) April 10.
Courants de Haute Frequence et Oscillations Electriques-Telegraphie sans Fil. Paul Janet. Experiences et Appareils de M. Ducretet. (32) Feb., 1899.

Marine.

The Institution of Naval Architects. Meeting. (11) April 7.
Cooper and Wigzell's Sounding Apparatus. Charles Bright. (11) April 7.
H. M. S. *Spartiate*. (12) April 7.
The Steam Trials of H. M. S. *Amphitrite*. (11) April 14.
The Naval Boiler of the Future. (12) April 14.
The Action of Oil on Waves at Sea. (12) April 14.
A Model Cruiser. (12) April 14.
Some Notes in Kronstadt, Russia. (12) April 14.
Launching Torpedoes in Rough Sea. (53) Feb. 17.
Saloon Steamer *Genève*. (48) April 1.
Experiments with a Marine Brake. (28) April 7.
The Best Water-Tube Boiler for the Navy. J. K. Robinson. (10) May, 1899.
Steam Consumption of Auxiliary Engines in Warships. (11) April 21.
The Ice-Breaking Steamer *Ermak*. (19) May 6.
Les Remorqueurs *Infatigable* et *Taillebourg*. M. Hachebet. (33) April 8.
Résistance á l'Avancement des Bateaux et Ondes Transversales. F. Chaudy. (32) Feb., 1899.

Mechanical.

Engine Specifications. R. H. Thurston, M. Am. Soc. C. E., and W. H. Bryan. (14) April 15.
Engine for the Coventry Tramways. (12) April 14.
Central Steam Condensing Plants. (20) April 20.
The New 2 000 H.-P. Engines and Alternator at Deptford. (12) April 21.
Vertical Electricity Supply Engines. (12) April 21.
A 650 H.-P. Gas Engine Direct Connected to a 520 K. W. Electric Generator. (13) May 4.
The Relation of Load to Steam Consumption in Steam Engines. E. Meyer. (48) April 8.
Superheated Steam, Its Generation and Use. (50) April 15.
Steam Pipes. J. T. Milton. (11) April 7; (14) April 15.
Stresses in Pipes Bent at Right Angles Caused by Heating to the Temperature of Steam at Various Pressures. T. Donaldson. (63) Part I, 1899.
Water-Tube Boiler for Motor Cars. (11) April 14.
Engines and Belleville Boilers of H. M. S. *Spartiate*. (12) April 7.
The Use of Coal Dust as Fuel for Boilers. Count Caracristi. (22) April 14.
Test of a Fan and Engine. S. S. *Belgia*. (14) May 6.
Cupola Blowers, Built by the Elsas Machine Co. in Mülhausen. F. Laurey. (48) April 15.
Blowing Engine at the Krompach Works (Bohemia). (12) April 21.
Test of a Large Pumping Engine, Pittsburg, Pa. (18) April 29; (13) May 4.
A New Air Lift Pump. (12) April 28.
Performances of Pumping Machinery; Clyde Trustees' No. 3 Graving Dock. (12) April 28; (22) April 28.
Propeller Pumps. (14) April 22.
Pumping Plant at Haughhead Colliery. (12) April 7.
Joel Electric Motor Carriage. (26) April 7.
The Efficiency of the Bicycle. Robert H. Fernald. (1) March, 1899.
Columbia Automobiles. Thomas B. Booth. (27) May 6.
The Mechanical Equipment of the Southern Station (Boston). Henry J. Conant. (7) March, 1899.
The James River, Va., Water Power Development. William Todd. (27) May 6.
Power Transmission Plant in the Maine Woods. (27) April 29.
Refrigerating Plant, System Pictet. N. Bräutigam. (48) Feb. 4.
Hot Blast Stove, Eston Steel Works. (12) April 28.
The Foundry Industry at the End of the Nineteenth Century. A. Ledebar. (48) April 22.
The Application of Blast Furnace Gases to Gas Motors, with Tests of a 60 H.-P. Gas Engine. E. Meyer. (48) April 22.
The Use, Design and Manufacture of Iron in Ornament. Chester B. Albree. (20) April 27.
Charging Steel Furnaces. (22) April 28.
The Testing of Machinery Castings. (22) April 14.
Strains in Hardened Steel of Large Cross Section. (50) April 1.
Note on the Endurance of Steel Bars Subjected to Repetitions of Torsional Stress. E. G. Coker. (63) Part I, 1899.
Progress in Rolling. (50) Jan. 15.
"Elio" Metal. (12) April 28.
The Extraction of Nickel from Its Ores by the Mond Process. W. C. Roberts-Austen. (63) Part I, 1899.
Cast-Iron Test Bars. Thomas D. West (11) April 28.
The Thermal Analysis of Steel. C. A. P. Turner (11) April 28.
Phosphor-Bronze—A Description of Its Characteristics and Method of Manufacture. L. L. Smith. (25) May, 1899.
The Connection Between Mechanical Engineering and Modern Shipbuilding. (22) April 28.
International System for Screw Threads. (57) April 6.

Mechanical—(Continued).

A Heavy Gun Lathe. (11) April 28.
Engineering Fallacies—Liquid Air as a Motive Power. (10) May, 1899; (46) April 29.
33-Ton Block Loading Titan Crane. (11) April 14.
Coke Ovens. Dr. V. Bauer. (50) April 15.
The Manufacture of Locking-Bar Jointed Pipe. (22) April 14.
Machine Corliss Compound de 500 Chevaux. (34) April, 1899.
L'Acetylene et ses Applications. E. Hubon. (32) Feb., 1899.
Gazogène Riché, à Distillation Renversée. F. Manaut et L. Roman. (32) Feb., 1899.
Moteur à Gaz a Double Effet et à surcompression Variable par Régulateur (Système Letombe). (33) April 1.
Notes Relatives à la Fabrication des Tubes et des Corps Creux en Fer ou en Acier, sans Soudure. M. Vinsonneau. (37) March, 1899.
Dispositif Applicable aux Générateurs de Vapeur. (36) April 25.

Military.

Explosion of a Gun at Sandy Hook, New York Harbor. (11) April 14.
Modern Machine Guns, Rifle Caliber. William Ledyard Catheart. (10) May, 1899.
The Mine Defence of Santiago Harbor. Lieut. E. E. Capehart, U. S. N. (19) May 6.

Mining,

Dredging for Gold. A. W. Robinson, M. Am. Soc. C. E. (10) May, 1899.
Our Coal Supplies. T. Forster Brown. (29) April 28; (22) April 28.
Shaft Sinking under Water at Lake of the Woods. (16) May 6.
Acetylene Gas as a Mine Illuminant. (22) April 28.
Electrical Transmission of Power in Mining. W. B. Esson. (63) Part I, 1899.
The Effect of Subsidence, due to Coal-Workings, upon Bridges and other Structures. S. R. Kay. (63) Part I, 1899.
Methods of Mining Ore in the Lake Superior Region. Nelson P. Hulst. (58) Feb., 1899.
The Douglas Coal Field, Lanarkshire. Robert Weir. (59) Vol. XX, Part 3.
Two Types of Electrical Coal Cutters. Thomas H. Barr. (59) Vol. XX, Part 3.
The Utica Mine Chlorination Plant. Thomas N. Smith. (16) April 22.
The Asphalt Deposits of Middle Park, Colo. H. A. Lee. (16) April 22.
A Prospector's Density Rule. J. Holms Pollok. (16) April 22.

Municipal.

Some of the Larger Transportation Problems in Cities. Edward E. Higgins. (3) May, 1899.
The Management, Maintenance and Cost of Public County Roads in Ireland under the Irish Grand Jury System. R. B. Sanders. (63) Part I, 1899.

Railroad.

Express Passenger Locomotive for the French State Railways. (11) April 21.
Vauclain Fast Passenger Compound Locomotive, Chicago, Burlington and Quincy Railroad. (25) May, 1899.
New Coupled Express Engines on Great Western Railway. Charles Rous-Marten. (12) April 28.
High Speed Passenger Locomotives. Clement F. Street. (61) March 25; (13) May 4.
Compound Locomotives on the Northern Pacific Railway. Edwin M. Herr. (61) March 25; (18) April 22.
A Chapter in Locomotive Stacks, Union Pacific Railway. (25) May, 1899.
Shop Testing of Locomotives in the United States. Richard A. Smart. (10) May, 1899.
Locomotive Design—The Working Strength of Materials. F. J. Cole. (25) May, 1899.
Consolidation and Mogul Locomotives, Southern Pacific Company. (25) May, 1899; (18) May 6.
The New Passenger Engines of the Burlington. (15) May 5.
Consolidation Locomotive, Lehigh Valley Railroad. (18) May 6.
Compound Consolidation Locomotives for the Southern Pacific. (15) May 5.
Heavy Locomotives for the Lake Shore. (40) April 28; (18) April 29.
American Locomotives for the Midland Railway, England. (18) April 29; (13) May 4; (40) May 5.
New Express Engine. (57) Jan. 12.
Electric Express Locomotives on the Paris-Lyons-Mediterranean R. R. O. Walzel. (53) Feb. 10.
A Gasolene Motor Car. (15) May 5.
The Hoadley-Knight Auto-Trucks. (20) May 4.
80,000-Pound 35-Foot Coal Cars, Lake Shore & Michigan Southern Railway. (25) May, 1899.
Ramsbottom Water Service for Moving Railroad Trains. Kemmann. (49) IV—VI.
A German Rail Joint. (12) April 28.
The MacPherson Switch and Frog, Canadian Pacific Railway. (18) April 15.
The Shield Used in Constructing the Orleans Railway Tunnel, Paris. (13) May 4.
The Development of Railroad Building from 1848 to 1898. Wilhelm Ast. (53) April 21.
South African Railway Development. Theodore Rennert. (10) May, 1899.
Revised English Train Rules. (15) April 14.
Resistance of Empty and Loaded Coal Cars. (15) April 14.
A New Device for Preventing the Sliding of Car Wheels by Air Brakes. (13) April 20.
Rules for Railway Location and Construction, Northern Pacific Railway. (13) April 20.

Railroad—(Continued).

Railways in Japan. (11) April 14.
Proposed Abolition of Grade Crossings at Syracuse. (15) April 21.
A Formula for Train Resistance. Henry Graftio. (17) May, 1899.
Parallel in the Development of the Locomotive and Electric Truck. John A. Brill. (17) May, 1899
The Cape to Cairo Railroad. H. G. Prout, M. Am. Soc. C. E. (15) April 14.
The Congo Railway. L. Trouet. (63) Part I. 1899.
The Pahartali Locomotive and Carriage Works. Assam-Bengal Railway. C. F. Bamford. (63) Part I. 1899.
The Standard Code of Train Rules of the American Railway Association (for single track). (15) April 21.
Standard Specifications for Interlocking. L. S. & M. S. Ry. (18) April 29.
Electric Headlights and Front End Signals. (40) May 5.
The Track Circuit. C. C. Anthony. (27) April 29.
Signaling on Single Line Railways. (26) April 21.
Main Air Pipe and Connections for Pneumatic Signals and Switches. A. M. Keppel, Jr. (15) April 21.
Inspection of Interlocking Plants. W. H. Elliott. (15) April 21.
The Jersey City Coaling Station of the Erie Railroad. (15) April 14.
The New Railway Stations at Omaha, Neb. (13) May 4.
New Union Passenger Station at Albany, N. Y. (18) May 6.
Long Island Railroad Improvements. (15) May 5
Depression of Philadelphia & Reading Railway Tracks in Philadelphia. (18) April 29.
Experimental Grain Elevators at the Hamburger Railroad Station in Berlin. Bork. (49) IV–VI.
Central Railroad Station in Osnabrück. Bergmann. (49) I–III.
English Electric Railroads. Albert H. Bridge. (15) April 21.
Generating and Distributing Electric Power for the Glasgow Corporation Tramways. H. F. Parshall. (13) May 4.
The Electrical Inspection of Street Car Equipments. Albert B. Herrick. (3) May, 1899.
Electric Railways in Paris. (17) May, 1899.
Some Details of the Murphy Third Rail System. (27) May 6.
The Electric Railroad between Stausstad and Engelberg. (48) April 15.
Electric Motive Power on Main Lines. C. P. Feldmann. (48) Feb. 18.
Le Mode de Pose de la Voie des Chemins de Fer dans les Souterrains. Jules Michel. (38) April, 1899.
Note sur une Distribution Spéciale Appliquée à la Compagnie du Chemin de Fer du Nord de l'Espagne. F. Panoux. (38) April, 1899.
Types Récents de Voitures et de Wagons des Chemins de Fer de l'État-Belge et Matérial de Tramways A. Morizot. (38) April, 1899.
Les Wagons à Intercirculation de la Compagnie de l'Est. Daniel Bellet. (36) April 10.
Le Chemin de Fer de la Jungfrau. Henry Martin. (33) April 22.
Le Chemin de Fer de Sfax à Gafsa (Tunisie). (33) April 22.
Machine à Poser les Voies de Chemins de Fer. (34) April, 1899.
Platforme Électrique à deux vitesses de l'Exposition de 1900. (Historique des tentatives et des applications de la locomotion par entrainement continu jusqu'à la). Armengaud, jeune. (32) Feb., 1899.
Tramways Électriques de Tours. Emile Dieudonné. (36) April 25.
Perfectionnement à apporter dans la construction des Voitures à Voyageurs. (36) April 25.

Sanitary.

Refuse Destruction. (11) April 7.
Refuse Furnaces. G. Watson. (63) Part I, 1899.
The Horsfall Destructor. (26) March 31.
Refuse Cremation at Edinburgh, Bradford and Oldham. (14) May 6.
Karachi Sewerage Works. J. Strachan. (63) Part I, 1899.
Sewer Repairs, Cincinnati, Ohio. (14) April 15.
Street Cleaning in Washington, D. C. (14) April 22.
The Sewerage of London, Ont. (14) April 22.
Experience in Sewer Construction. L. M. Hastings. (1) March, 1899.
Maintenance of the System of Separate Sewers at Newton, Mass. Stephen Childs, M. Am. Soc. C. E. (1) March, 1899.
Street Cleaning in San Francisco. (14) May 6.

Structural.

The Vulcanite Portland Cement Company's Works. F. H. Lewis, M. Am. Soc. C. E. (14) May 6.
Gravity Steam-Heating Problems. (14) May 6.
The Sutro Baths of San Francisco. F. W. Parsons. (46) April 29.
The New Foundry of the General Electric Company. (13) April 13; (14) April 15.
Heating the Foundry of the General Electric Company. (20) April 27.
The Employment of Electrical, Heating and Sanitary Engineers in Building Construction. (13) April 13.
The Calculation of I-Beam Grillage Foundations. J. B. Nau. (13) April 20.
Column and Beam Connections in the Williamson Building. (14) April 22.

Structural—(Continued).

The Wanamaker Stables, New York. (14) April 22.
"Electric" Steel Tank Elevator at Buffalo, N. Y. (18) April 8.
Expansion of Iron Structures. F. Z. Schellenberg. (62) April 21.
Repeated Stresses. Jerome Sondericker. (7) March, 1899.
The Planning and Building of the Southern Station, Boston. George B. Francis, M. Am. Soc. C. E. (7) March, 1899.
On the Theory of Domes and Steeples. H. Müller. (48) April 8.
General Assembly Hall and Auditorium of the Technical Highschool in Karlsruhe. (49) IV–VI.
Refrigerating Plant at City Market Hall in Vienna. J. Hermanek. (53) Jan. 6.
École Libre, Rue Bobillot, à Paris. (35) April, 1899.

Topographical.

Some Observations on the Use of Polar Planimeters. Walter W. Patch. (13) April 13.
Trial Survey for a Railway over the Outeniqua Mountains, Cape Colony. C. E. Stewart. (63) Part I, 1899.

Water Supply.

Standpipe for the City of Mülheim. F. Thometzek. (48) Jan. 28.
Machinery of the Water Works at Breitensee. C. Budil. (48) Jan. 7.
The Water Tower in Kiel. R. Schmidt. (51) Feb. 8.
Subaqueous Pipe Laying at Delray. (14) April 15.
Electrolysis in Battle Creek, Mich. W. W. Brigden. (14) April 15.
The Construction of High Earth Dams. (14) April 15.
Water Filtration, Zurich, Switzerland. James H. Fuertes, M. Am. Soc. C. E. (14) April 22.
The Amballa (India) Water-Works. (14) April 22.
Allowances for Depreciation in the Pipe System of the Los Angeles Water-Works. (13) May 4.
The New Water-Works of Camden, N. J. (13) May 6.
Nouvean Réservoir d'Eau de la Ville de Mülheim. (33) April 22.

Waterways.

The Economic Relation of the Proposed Deep Waterways to the State of New York. George W. Rafter, M. Am. Soc. C. E. (7) March, 1899.
Buenos Ayres Harbor-Works. (11) April 21.
The Rhein-Elbe Canal. (50) April 15.
New Harbor Improvements in Stettin. F. Krause. (49) I-III.
River Regulation Works and Harbor and Canal Construction in Germany. L. Franzius and G. H. de Thierry. (63) Part I, 1899.

NEW BOOKS OF THE MONTH.

Unless otherwise specified, books in this list have been donated to the Library by the Publisher.

DICTIONARY OF ELECTRICAL WORDS, TERMS AND PHRASES.

By Edwin J. Houston. Fourth Edition. Cloth, 10 x 7 ins., 990 pp., illus. New York, The W. J. Johnston Company, 1898. $5.00. (Donated by *The Electrical World and Engineer.*)

The rapid growth of electrical science and the almost daily addition of new terms and phrases have led to the production of an electrical vocabulary that is bewildering in its extent. In order to leave no doubt concerning the precise meaning of the terms and phrases defined, the plan has been adopted by the author, of giving: (1) A concise definition. (2) A brief statement of the principles of the science involved in the definition; and (3) where possible and advisable a cut of the apparatus described or used in connection with the term or phrase defined.

STANDARD TABLES FOR ELECTRIC WIREMEN.

With Instructions for Wiremen and Linemen, Underwriters' Rules, and Useful Formulas and Data. By Charles M. Davis. Fourth Edition. Roan, 5 x 3 ins., 128 pp. New York, The W. J. Johnston Company, 1896. $1.00. (Donated by *The Electrical World and Engineer.*)

The wiring tables have been recalculated for this edition and a section on the calculation of alternating current wiring added.

CENTRAL STATION BOOKKEEPING

And Suggested Forms; with an Appendix for Street Railways. By Horatio A. Foster, M. Am. Inst. E. E. Cloth, 9 x 6 ins., 139 pp., tables. New York, The W. J. Johnston Company, 1896. $2.50. (Donated by *The Electrical World and Engineer.*)

The author has endeavored to show such a classification of accounts and a system of reports for central electric light and power stations as to enable the management by their use to know the full details of the business of distributing electric current. The contents are: Customers' Ledgers; Objects of Accounting and Classification; Operating Expense; Income from Operation; Profit and Loss; Property Accounts; Liability Accounts; General Classification; General Books; Monthly Summary Statements; Bookkeeping for Electric Street Railways.

ALTERNATING CURRENTS OF ELECTRICITY.

Their Generation, Measurement, Distribution and Application. By Gisbert Kapp. American Edition. Cloth, 7 x 5 ins., 166 pp. New York, The W. J. Johnston Company, 1896. $1.00. (Donated by *The Electrical World and Engineer.*)

The author assumes that the reader is acquainted with the behavior of steady currents in circuits whose constants are readily obtainable, and treats briefly of periodic currents in general, the phase relations of impressed and induced electromotive forces possible in simple circuits, alternators, the requirements of central stations, alternating current motors and multiphase currents. There is an index of eight pages.

ALTERNATING CURRENTS.

An Analytical and Graphical Treatment For Students and Engineers. By Frederick Bedell and Albert Cushing Crehore. Third Edition. Cloth, 9 x 6 ins., 325 pp. New York, The W. J. Johnston Company, 1895. $2.50. (Donated by *The Electrical World and Engineer.*)

The authors have endeavored to meet the demand for definite information in regard to the fundamental principles governing the flow of variable or alternating currents. Their purpose has been to use such mathematical terms and analytical methods as make it possible for the demonstration to be exact and rigorous and at the same time to express the results in such a way as to be intelligible to those who do not desire to follow the methods of proof but are only interested in the conclusions reached. The subject is divided into two parts. Part 1. Analytical Treatment. Part 2. Graphical Treatment.

RECENT PROGRESS IN ELECTRIC RAILWAYS.

Being a Summary of Current Periodical Literature Relating to Electric Railway Construction, Operation, Systems, Machinery, Appliances, etc. Compiled by Carl Hering. Cloth, 7 x 5 ins., 389 pp., illus. New York, The W. J. Johnston Company, 1897. $1.00. (Donated by *The Electrical World and Engineer.*)

The compiler, feeling that the volume of current electrical literature is so great that it is almost if not quite impossible for one to keep satisfactorily informed of the advances made in the different branches of electrical engineering, has issued a *résumé* of articles upon electric railways appearing in periodicals. He has assumed no responsibility for any of the statements compiled. The contents are: Historical Notes; Development and Statistics; Construction and Operation; Cost of Construction and Operation; Overhead Wire Surface Railways; Conduit and Surface Conductor Systems; Storage Battery Systems; Underground (Tunnel) Systems; High Speed Interurban Railroads; Miscellaneous Systems; Generators, Motors and Trucks; Accessories.

HYDRAULIC MACHINERY.

With an Introduction to Hydraulics. By Robert Gordon Blaine. Cloth, 9 x 6 ins., 383 pp., illus. New York, Spon & Chamberlain, 1897. $5.00.

In this work the author has made use of elementary applications of the calculus. The book was written with the hope of assisting students to obtain a fairly thorough groundwork in the knowledge of hydraulic power. The headings of chapters are: Introduction; Compressibility of Water; The Hydraulic Press; Hydraulic Jack; Applications of the Hydraulic Press; Fluids and Fluid Pressure; Lines of Force and Equipotential Surfaces; Hydraulics—Friction of Water at Different Velocities; Flow of Water—From Orifices—Through Pipes—In Channels; Coefficients of Hydraulic Resistance; Distribution of Energy; The Measurement of Flowing Water; Jet Propulsion; Nozzles and Jets; Water-Wheels; Centrifugal Pumps; Turbines; Speed Regulation; Hydraulic Transmission of Power; Hydraulic Cranes; Hydraulic Lifts; Hydraulic Canal Lifts and Graving Docks; Hydraulic Engines; Bridge and Dock-Gate Machinery; Hydraulic Machinery on Board Ships; Hydraulic Machine Tools; Pumps; The Hydraulic Intensifier; Hydraulic Rams; Hydraulic Brake; Waste of Power in Hydraulic Mains. There is an index of six pages.

MODEL OF A HORIZONTAL STEAM ENGINE.

Furnished with Meyer's Variable Expansion Gear; with a Brief Description of the Parts and the Method of Working, and a Discussion and Explanation of Zeuner's Valve Diagrams. By Chr. Volkert, tr. and ed. by Arnold Philip. Boards, 7 x 10 ins., 25 pp. plates. London, 1896, George Philip and Son. (Imported by Thomas Whittaker, New York). $1.00.

The author has endeavored to make clear to persons other than engineers, who are indirectly more or less interested in steam engines, a knowledge of their working. He has taken as a model, the type in which the periodic distribution of steam is regulated by slide-valves. The contents are as follows: General Arrangement of Parts; Description of the More Important Parts of the Steam Engine; Steam Distribution by Means of Ordinary "D" Slide-Valves. The book is illustrated with dissected and colored drawings.

STEAM BOILER PRACTICE.

In Its Relation to Fuels and Their Combustion, and the Economic Results Obtained with Various Methods and Devices. By Walter B. Snow. Cloth, 9 x 6 ins., 297 pp. New York, John Wiley & Sons, 1899. $3.00.

Steam boiler practice as here treated is concerned rather with the results obtained than with a detailed description of the methods and devices used to secure the economical combustion of fuel in connection with a steam boiler. The author has endeavored to indicate the possible gain or loss to result from a given arrangement and to point the way toward securing the highest efficiency in steam boiler practice. Contents are as follows: Steam Boiler Practice; Water and Steam; Combustion; Fuels; Efficiency of Fuels; Efficiency of Steam Boilers; Rate of Combustion; Draft; Chimney-Draft; Mechanical Draft; Rules for Conducting Boiler-Trials. There is an index of four pages.

LOCOMOTIVE ENGINE RUNNING AND MANAGEMENT.

Showing How to Manage Locomotives in Running Different Kinds of Trains with Economy and Dispatch; Giving Plain Descriptions of Valve Gear, Injectors, Brakes, Lubricators, and other Locomotive attachments; Treating on the Economical Use of Fuel and Steam; and Presenting Valuable Directions about the Care, Management, and Repairs of Locomotives and Their Connections. By Angus Sinclair. Cloth, 8 x 5 ins., 438 pp., illus. 21st Edition. New York, John Wiley & Sons, 1899. $2.00.

The author's aim has been to treat all subjects discussed in such a way that any reader would clearly understand every sentence written. No attempt is made to convey instruction in anything beyond elementary problems in mechanical engineering. The construction and management of compound locomotives are not described, the author confining himself to the single expansion engine. A chapter is devoted to the examination of firemen for promotion. There is an index of twelve pages.

THE STORY OF THE RAILROAD.

By Cy Warman. Cloth, 8 x 5 ins., 280 pp., illus. New York, 1898, D. Appleton & Company. $1.50.

This book is a chronicle of the life which began with and accompanied the building of a transcontinental highway. It pictures the construction of the earlier lines across the deserts and mountains of the Western United States. Prominence is given to the Union Pacific, but in a sense, the Atchison, Topeka and Santa Fé is the typical railroad of the book.

THE NEW ROADMASTER'S ASSISTANT.

A Manual of Reference for those having to do with the Permanent Way of American Railroads. By George Hebard Paine. Roan, 7 x 4½ ins., 259 pp., illus. *The Railroad Gazette*, 1898. (Donated by the author.)

This volume is the successor to the Roadmaster's Assistant, written in 1871 and revised in 1877. It aims to be practically useful and theoretically correct. Its scope in the treatment of problems arising in the maintenance of way on railroads is indicated by the chapter headings, which are as follows: General Remarks; Organization and Methods of Work; Fences, Highway Crossings and Platforms; Miscellaneous Fixtures and Station Grounds; Water Supply; Drainage; Culverts, Trestles and Bridge Floors; Ballast; Cross Ties; Rails and Fastenings; Track Work; Tools; Frogs, Switches and Switch Stands; Emergencies and Train Signals; Fixed Signals; Rules and Tables. There is an index of five pages.

RAILWAY ECONOMICS.

By H. T. Newcomb. Cloth, 8 x 5 ins., 152 pp. Philadelphia, Railway World Publishing Co., 1898. $1.00.

This work is intended to present the principal facts which establish the position of railway transportation in the present organization of industry in the United States. The contents are: Transportation; Railway Service in the United States; Capitalization; Income and Expenditure: The Decline in Charges; Rates and Prices; Causes of the Decline in Charges; Competition Among Railways and the Decline in Charges; Competition Among Producers and the Decline in Charges; The Law of Increasing Returns; Increasing Use of Railway Facilities; Socially Profitable Transportation; How Socially Unprofitable Transportation May Occur; Should the Business of Railway Transportation be Remunerative? Energy Expended on Joint Account; Relatively Reasonable Rates; Unjust Discrimination in General; Consolidation; Pooling; Associations; Taxation; Construction.

SEWER DESIGN.

By H. N. Ogden. Cloth, 8 x 5 ins., 234 pp.; illus.; 5 plates. New York. John Wiley & Sons, 1899. $2.00.

This work represents a course of lectures, given in the College of Engineering, Cornell University, treating only of the subject of sewer design. The contents are: General Considerations; Preparatory Maps and Data; Excessive Rains; Proportion Reaching the Sewers; Relation of Density to Percentage; Mathematical Formulas; Estimating Future Population; Amount of Sewage per capita; Ground Water Reaching Sewers; Grades and Self-Cleansing Velocities; Development of Formulas for Flow; Kutter's Formula; Sewer Diagrams; Use of Diagrams; Sewer Plans; Sewer Cross-Sections; Flushing; Use of Tanks.

LANDSCAPE GARDENING.

As Applied to Home Decoration. By Samuel T. Maynard. Cloth, 8 x 5 ins., 338 pp., illus. New York, John Wiley & Sons, 1899. $1.50.

The author has endeavored to supply information which will enable persons in limited circumstances to properly care for ornamental trees, shrubs, or plants, and to show how to group and combine them with the lawn, the dwelling and other buildings, or houses adjoining. Headings of chapters are: Landscape Gardening and Home Ornamentation: Ornamenting New Homes; Preparation of the Land; Trees—Their Planting and Care; Shrubs, Hedges and Hardy Climbers; Walks and Drives; Renovating and Improving Old Homes; Country Roads and Roadside Improvements; Parks, Public Squares, School Yards, etc.; Description of Trees; Evergreen Trees; Ornamental Shrubs; Herbaceous Plants, Aquatic Plants, Insects injurious to Ornamentals; The Home Fruit Garden. There is an index of twenty-four pages.

ELEMENTS OF WATER SUPPLY ENGINEERING.

By E. Sherman Gould. Cloth, 9 x 6 ins., 168 pp. New York, The Engineering News Publishing Co., 1899. $2.

All matter previously published in the first and second editions of "Practical Hydraulic Formulæ," in 1889 and 1894, is contained in this work; but in the present volume, under the heading of "Notes to Parts I and II," material has been added supplementing and extending many points. An entirely new part (Part III), has been added. The contents are: Flow of Water Through Masonry Conduits; Some Details of Tunnel and Aqueduct Construction; Filtration of Public Water Supplies; Pumping Engines and Duty Trials; Arches and Abutments. The work contains a number of hydraulic tables and an index of six pages.

LOUISVILLE FILTRATION EXPERIMENTS.

Report on the Investigations into the Purification of the Ohio River Water at Louisville, Kentucky; Made to the President and Directors of the Louisville Water Company. By George W. Fuller. Cloth, 12 x 9 ins., 461 pp., plates. New York, D. Van Nostrand Company, 1898. $10.00. (Donated by the author.)

This is a report of tests and investigations which had for their purpose the determination of the quality of the river water after purification on a practical scale by filters, or systems of purification, and the collection and compilation of such data as would indicate the cost of construction and operation of these filters or systems. There is an index of eight pages.

STEAM HEATING FOR BUILDINGS.

Baldwin on Heating; or, Steam Heating for Buildings, Revised—being a Description of Steam Heating Apparatus for Warming and Ventilating Large Buildings and Private Houses, with Remarks on Steam, Water, and Air, in Their Relation to Heating; to which are added Useful Miscellaneous Tables. By William J. Baldwin. Fourteenth Edition, Revised and Enlarged. Cloth, 8 x 5 ins., 365 pp.; illus., 6 plates. New York, John Wiley & Sons, 1897. $2.50. (Donated by the author.)

In the present edition, so far as the work relates to the principles of steam heating, where the water of condensation is returned to the boiler by gravity, little change from preceding editions has been made; but to bring it to modern practice in the use of steam by other methods, a general revision was necessary. An endeavor has been made to confine the work to facts as developed by practice, experience and experiment and comment and opinion are refrained from, except where the lack of an explanation might lead to error or misunderstanding. There is an index of eleven pages.

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PAPERS AND DISCUSSIONS.

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PILE-DRIVING FORMULAS: THEIR CONSTRUCTION AND FACTORS OF SAFETY.

By Charles H. Haswell, M. Am. Soc. C. E.

To be Presented September 6th, 1899.

Supporting Piles.

Experiments on the impact of the ram of a pile-driver, from the ordinary heights of its operation, and deductions therefrom, are not as satisfactory as the importance of the subject requires, as evidenced by the variety in the elements of computation, formulas and factors of safe loads.

Authors upon physical laws, and engineers who have essayed to give a formula for the impact and effect of a ram on a pile, vary not very materially, but in some instances have introduced elements having little or no connection with it, and consequently of no essential value: as the diameter, weight and length of a pile, its mean depression per number of blows, its modulus of elasticity, and the cube root of the height of fall of the ram.

The resistance a pile offers to the blow of a ram is the measure of its capacity to sustain a stress, wholly independent of its diameter, weight, length, modulus of elasticity, etc.

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Thus, the diameter of a pile and the distance of its penetration are not proper elements in the computation, as their effect is embraced in their limitation of the penetration of the pile; added to which the weight of the pile is of consideration only as involving a weight of ram proportionate to its inertia, and the relative elasticity of a pile is of little moment, for when a pile approaches to the penetration of required resistance or refusal, if its head is broomed it is dressed off, and if split or liable to be, it is confined by a ring. In fact, the weight and fall of a ram being proportionate to a pile, all other elements, except that of penetration, are rendered of no consideration, as elements of the computation of effect; and particularly when so great a factor of safety as the varied conditions under which a pile is driven requires, and which is given by various authors of formulas as ranging from 0.083 to 0.33, omitting Weisbach's extreme of $\frac{1}{1000}$.

In the following treatment of the subject, a pile is considered as driven to a final depression of 0.5 in. or less, with a level and sound head, as far as the operation renders it practicable; and when the weight of the superstructure to be sustained requires the number of piles to be increased proportionately thereto, this increase objectionably involves the necessity of placing them at less than a proper distance apart, for when they are driven close in a moist or clayey soil, the last one driven will, by the lateral compression, cause the adjoining ones to rise.

In the formulas of Major Saunders, U. S. A., Molesworth, Weisbach, Mason, Trautwine, Nystrom, Wellington and the "Dutch," the final depression or penetration, either in inches, feet or *pro rata*, is given as a divisor. Hence, assuming that the impact of a ram in a given case is 100 000 lbs., and the depression is 0.5 in., $100\,000 \div 0.5 = 200\,000$ lbs., and if it is 0.25 in., $100\,000 \div 0.25 = 400\,000$ lbs.; and in like manner to infinity or positive refusal; and yet the impact would not have increased, as the effect of the blows is not cumulative. The resistance of a pile is not measured by a summation of the several impacts it has received by the fall of the ram, or by the *pro rata* of its depression per number of blows, as by the "Dutch" formula. When a street paver depresses a stone 1 in. with his rammer, and 0.5 in. at a succeeding blow, he has not correspondingly increased the force or effect of it, neither are the blows of a carpenter in driving a bolt cumulative, for only the last blow gives the measure of the impact on

it. Hence, the effect of the blow of a ram, however apparent to observation, is lost to consideration when the one that follows it occurs, as the measure of the last is the measure of the resistance the pile offers, and that resistance is the single sustaining stress or weight that the pile will bear.

The matter, then, is not how the resistance of a pile is obtained, not what the weight of it, not what its inertia or the effect of its cushioning, but what was the power required to drive it to its final depression, or what is the measure of its resistance to stress or to sustaining a weight?

Illustration of the Several Formulas Referred To.

Assume rams having weights of 1 000 and of 2 000 lbs., falling, respectively, 20 and 25 ft., which are taken as exponents of ordinary practice. Mean section of pile, 100 sq. ins.; weight, 1 000 lbs.; surface, 130 sq. ft.; length, 35 ft.; and penetration under last blow, 0.5 in. = 0.043 ft.

(1) *Major Saunders.* $\dfrac{R\,h \div d}{8} = W.$

R representing weight of ram and W safe load, both in pounds; h = height of fall, and d = depression of last blow, both in feet.

(2) *Rankine.* $\sqrt{\left(\dfrac{4\,E\,s\,W\,h}{l}\right)} + \dfrac{4\,E^2\,s^2\,d^2}{l^2} - \dfrac{2\,E\,s\,d}{l} = L.$

E = modulus of elasticity, s = section of pile in square inches, l = length of pile in feet, and L = dead load without further depression. Other symbols as preceding. Factors of safety, 5 to 1.

This formula embraces modulus of elasticity, length and sectional area of piles, all of which, for the causes here given, are held to be an unnecessary complication and of no practical value.

A Second Formula by Rankine. $A\ 1\,000 = L$ at refusal of pile; and when not, $A\ 200 = L$; or a factor of safety of 5.

A = mean area of section of pile in square inches, and L = load in pounds.

(3) *Molesworth.* $\dfrac{W\,H}{8\,D} = L.$

W = weight of ram, and L = safe load, both in hundredweights. H = height of fall, and D = set of pile by last blow, both in inches.

(4) *Mason.* $\dfrac{W^2\,h}{S\,(W+p)}$ = extreme load.

S = depression by last blow, in feet; p = weight of pile in pounds; other symbols as preceding. Factors of safety, 10 to 4.

(5) *Trautwine.* $\frac{\sqrt[3]{h\ W\ 0.023}}{s+1} \times 2\,240 =$ extreme load. s = depression by last blow in inches; other symbols as preceding. Factors of safety, 10 to 2.

(6) *Nystrom.* $\frac{M^3\ S}{6\ s\ (M+m)^2} =$ safe load.

M = weight of ram, and m = weight of pile, both in pounds. S = fall of ram and s depression, both in inches; other symbols as preceding.

(7) *Wellington.* $\frac{2\ Wh}{s+1} =$ extreme load.

s = depression by last blow, in feet; other symbols as preceding.

(8) *McAlpine.* $80\ (W + 0.228 \surd F - 1) = X.$

W = weight of ram, in tons; F = fall of ram, in feet, and X = load at refusal. Factor of safety, 0.33.

TABLE No. 1.—Recapitulation of Results. Depression in all Cases, 0.5 in.

Formulas.	Factors, as given by the several Authors.	1 000 Lbs. Falling 20 Ft.		2 000 Lbs. Falling 25 Ft.	
		Load.	Safe load.	Load.	Safe load.
		Pounds.	Pounds.	Pounds.	Pounds.
1. *Saunders*	8	476 000	59 500	1 190 000	148 750
2. *Rankine*	5 to 1	332 125	66 425	646 246	129 249
3. *Molesworth*	5	100 000	20 000	200 000	40 000
4. *Mason*	8	480 080	60 010	1 199 520	149 940
5. *Trautwine*	10 to 4	273 598	27 360 68 399	861 326	86 133 21 533
6. *Nystrom*	12 to 2	93 218	7 768 46 609	200 592	16 716 100 296
7. *Wellington*	6	166 086	27 681	658 434	109 739
8. *McAlpine*	3	83 328	38 387	184 934	95 970
			27 776		61 644
Mean of results	5.75	250 554	40 901	645 131	87 271

* A mean by the author.

The varying figures in the fourth and sixth columns, under safe load, represent results as computed by the formulas of different authors, with their factors.

The following formulas were not included in the preceding recapitulation and summation of results, as the factors in the first are held

to be too extreme for practical application, and the introduction in the other, of the average penetration per blow, per 100 blows of the ram, is not an element of any value; for, as before stated, the final blow of a ram is the only one that measures its effective value.

Weisbach.—One of Weisbach's formulas embraces elements similar to those of Rankine, with a factor of safety of 10 to 100. A second,

$$\frac{w^2 h}{s(w+p)} + (w+p) = \text{extreme load.}$$

w = weight of ram, and p = weight of pile, both in lbs.; s = penetration by last blow, in feet = 0.043; other symbols as in the other formulas. Factors of safety, 100 to 10.

"*Dutch.*" $$\frac{B^2 h}{(B+M)} e = W.$$

B = weight of ram, and M = weight of pile, both in lbs. e = average penetration per blow for 100 blows, and h = height of fall, both in inches. Factor of safety, 6.

In the "Dutch" formula, e represents the average depression of the pile per blow, per 100 blows of the ram, which is not only an element measurably impracticable of attainment in practice, but is wholly valueless and difficult of attainment except with the heavy ram and low fall practiced by the authors of the formula.

A recorded case furnishes the following illustration of the formula: B and M, respectively, 2 205 and 2 030 lbs. and e = 0.28 in.

$$\frac{2\,205^2 \times 20}{(2\,205 + 2\,030)\ 0.28} \div 6 = 13\,665 \text{ lbs.}$$

Operating this formula with the elements given in the first of the preceding cases, the result would be 35 715 lbs.

Factor of Safety.—In deciding upon the factor of safety the following elements are to be considered:

The friction of the guides of the leader and of the hoisting line of the ram in the sheave and over the drum (ascertained by experiment with a very heavy ram to be 0.2 ft. of penetration, but with a light ram it would be materially more); the resistance of the atmosphere to the fall of the ram and its cushioning on the head of a pile however squarely it may be dressed off; the want of verticality, both of the fall of the ram, and of the vertical plane of the pile and its consequent lateral vibration; the inertia and frequent splitting of the pile on a boulder; the vibration, if driven in a trench or with a "follower";

the condition of the soil, whether wet, moist, or dry, that is, if it is wholly dry soil, if the pile is embedded or partially exposed above ground or water, or is wholly immersed in wet soil or surface water, all of which are elements in the determination of the factor.

In this connection is to be considered the subsidence of the soil around a pile which has been temporarily arrested in its driving, the effect of which, in the course of a few hours under favorable conditions of soil, has been observed to approach to that of the resistance of a pile at its last blow.

The following are records of some ascertained sustained weights on piles:

1. At the London Bridge over the Thames,* the piles are reported to sustain 80 tons. Assuming them to have been driven under the extreme condition of a ram of 1 500 lbs. falling 20 ft., their sustaining power in excess of that given by the formula $W\sqrt{2\,g\,h}$ would be 23.94 tons, and $80 \div 23.94 = 3.33$ to 1.

2. A pile driven with a ram of 2 000 lbs. falling 5 ft. supported 47 tons, an excess over the formula of $16 \sim 47 = 31$ tons; and $47 \div 16 = 2.93$ to 1.

3. A ram of 1 700 lbs. falling 16 ft. supported 70 tons, or an excess of $24.3 \sim 70 = 48.7$ tons, or as 2.88 to 1.

4. A test pile by the United States Government, driven by a ram of 910 lbs. falling 5 ft. with a depression of 0.375 in. under the final blow, supported 26.6 tons, or an excess of $7.28 \sim 26.6 = 19.32$ tons, or 3.65 to 1.

The average of results for safe loads, deduced by the several formulas given are, for a ram of 1 000 lbs. falling as given, 40 901 lbs.; a result 2.04 times greater than that given by the formula of an author of highest recognized ability, and 1.46 times less than by another author of similar standing. For a ram of 2 000 lbs. falling as given, the average is 87 271 lbs.; a result 2.182 times greater than that given by the first author referred to, and 1.35 times greater than that given by the other.

In view of this variance, the author essayed to ascertain the difference between the results by the formula, or foot pounds, and the actual *vis viva* of a falling body; and although the experiments were necessarily on a very limited scale, the following results were obtained:

* "Trautwine's Engineer's Pocket-Book," pp. 643-644.

Weight.	Fall.	Velocity.	Impact.	Ratio of Impact to Velocity.
Pounds.	Feet.	Feet.	Pounds.	
1.....	1	8	32	4.0
1.....	1.5	9.5	42	4.3
1.....	2	11.31	52	4.6

Reviewing these and the various other elements submitted, it appears that all the formulas here illustrated, for piles driven under the usual requirements of practice, as to weight of ram, height of fall, character of soil and final depression, are but arbitrary, with factors of safety having the wide range of three-tenths to one-twelfth. They present even the varied conditions of the weight multiplied by the height, by its square, and also by its cube, while that of Wellington presents the exceptional condition of the weight being multiplied by twice the height of the fall; all of them being at variance with the accepted rule, that the velocity of the weight and its resultant impact is as $\sqrt{2\,g\,h}$. Hence, for heights of fall of 16, 25 and 36 ft., the product of the Wellington formula would be as 32, 50 and 72 ft., while the velocity of impact would be as 32, 40 and 48 ft.

Further, that the impact or *vis viva* of a body falling freely, as determined by the experiments here referred to, exceeds four times that given by the formula for its final velocity, which result is verified by the illustrations given of the actual support of piles noted, to which should be added an estimate of the frictional loss of the ram in operation.

Reviewing the elements presented, the following formula is submitted: $\frac{4\,w\sqrt{2\,g\,h}}{F}$, or $\frac{4\,w\,8\,\sqrt{h}}{F}$ = safe load in pounds.

w = weight of ram in pounds, and h = height of fall in feet.

F, or the factor of safety, in consideration of the several losses of effect recited, and especially that of the brooming of the head of a pile, and the driving of piles in trenches or with a "follower," is assumed at from 3 to 6, according to the nature and condition of the soil, the character of the piles and the integrity of their driving.

Applying the formula to elements similar to those given for the several formulas recited, the results would be, for the illustration of a ram of 1 000 lbs. falling 20 ft., with a factor of 3:

$$\frac{4 \times 1\,000 \times 8\,\sqrt{20}}{3} = L;$$

and for the ram of 2 000 lbs. falling 25 ft., eliminating the numerator 4, and correspondingly reducing the divisors, 3 to 6, to 0.75 and 1.5, the operation is simplified, thus:

$$\frac{1\,000 \times 8\sqrt{20}}{0.75} = 47\,701 \text{ lbs., and } \frac{2\,000 \times 8\sqrt{25}}{0.75} = 106\,666 \text{ lbs.;}$$

and with a factor which is a mean between 0.75 and 1.5 = 1.125, the results would be, respectively, 31 801 and 94 814 lbs., results in accordance with the observation of an exceptionally extensive and varied experience in the operation under consideration.

In further support of the formula submitted, the following is presented:

First.—That at the London Bridge, the sustaining capacity of the piles is reported to be 3.33 times the weight deduced by the ordinary formula for the impact of a falling body; in other cases to have been 2.93 and 2.88 times, and in a test case by the United States Government to have been 3.65 times, giving a mean of 3.1975; to all of which is to be added the greater or less losses of effect of the ram in falling, as herewith detailed.

Second.—That the test pile previously cited, which was driven and observed by officers of the United States Government, the ram weighing 910 lbs., the fall being 5 ft., and the depression of the pile at the final blow being 0.375 in., sustained a stress of 26.6 tons; that the sustaining power of the pile, deduced by the ordinary formula for the velocity of a falling body, would be $910 \times \sqrt{2gh} = 16\,457$ lbs. = 7.34 tons; and that these results of 7.34 and 26.6 are as 1 to 3.62, approximating closely to the 4 assumed in the formula presented.

Third.—That piles driven to sustain the vibratory stress of a bridge under a hammer of 1 200 lbs., falling 20 ft., with a penetration of 0.75 in., enduringly sustained 18 tons each, and by the formula the result is 17 tons.

Regarding the distance from centers at which piles should be driven; in sand or small gravel, this may be 2 ft., but for saturated sand and earth, or in silt, a greater distance is required, as small piles are liable to be disturbed in their position by the driving of a larger one adjoining them, and in some practice, in determining the capacity of a range of piles, it is proper to reduce the result obtained by the formula, to meet incidental conditions, such as negligence in driving, or in superintendence, the frequent and unobserved splitting or crushing of a pile on a stone or boulder.

The deductions and opinions here given are the result of observations of a very varied and extensive experience in pile-driving, in every variety of soil, encountering quicksand, boulders, submerged crib-work, stone filling, in water, and under the embarrassing conditions met in a trench, when the hammer was necessarily arrested many feet above the designed level of bearing of the piles, and for structures of various capacities and heights, some attaining twelve stories; and whenever practicable, a bench-mark was established and ultimate results observed.

Tenacity of Piles.—The effects of the tenacity were illustrated at Cuxhaven, England, with a fir pile, 1 ft. in diameter at the head, driven 15.7 ft. in fine and drifting dry sand; and which, at the termination of 23 days, required a power of 17 472 lbs. (7.8 tons) to withdraw it; also one of 17 ins. diameter, driven to a like depth, at the termination of 20 years, required a power of 57 120 lbs. (25.5 tons), and at 50 years, 98 000 lbs. (43.75 tons), to remove it.

Friction of Pine Sheet Piles in Clay.—The removal of a coffer-dam after 5 years' existence furnishes the following elements:

Piles of Memel fir, 12.5 ins. square, length 40 ft., driven to an average depth of 18.25 ft., the average superficial area in the soil being 76 sq. ft., the net resistance, after deducting the weight of the piles, etc., was 71 277 lbs., giving a coefficient or factor of 93.8 lbs. per square foot for the bearing of the clay on the inner and outer surface of the piles.

Follower.—Whenever a follower is used, however short it may be, the factor of safety should be increased; and when piles are driven in trenches below the driver and their final depression below the drop of the ram is so much as to involve the use of a long follower, the extreme factor should be used; and with piles of spruce or like wood, their heads should be ringed as soon as their penetration becomes difficult.

Brooming.—The result of brooming the head of a pile, in consequence of its resistance, as determined by experiment, was to increase the number of blows of a ram compared with a sound head from 1 to 1.49.

Sheet Piling.—Sheet piling would be much more effective, either in water or wet soil, if constructed of two or more thicknesses of timber laid together and lap broken for half the width of a piece if two are used and one-third the width if three are used, and fastened

with locust, or iron blunt bolts, the number of which and their distances apart being determined by the stress of the driving. The off edge of the foot of the pile should be cut or tapered to an extent which will tend to lead and retain it close to the adjoining pile.

Notes Deduced from Operations of Various Authors on Piling.—In wet sand, silt, or any light soil, piles should be driven rapidly, otherwise the subsidence of the material in which they are driven will furnish increased resistance.

In strong or stony soil, a heavy ram with a low fall is better than a light ram and a high fall. If the soil is very strong with stone, the piles should be shod.

Inasmuch as a stratum of light or wet earth may exist below a heavy one, before commencing piling upon a new soil, in order to be assured, not only of its consistency, but also of the depth of such condition, a test pile of small diameter should be driven to as great a depth as practicable.

The support of compressed sand is estimated at 85 lbs. per square foot, and the friction on the surface of a pile at 614 lbs. per square foot.

Finally, in a recent case, in consequence of the foundation of a building of eleven stories being 7 ft. below that of two exceptionally heavy adjacent structures of ten stories each, the walls of which were laid on a concrete bed over wet sand, piling was necessarily arrested at distances of 7 ft. from these adjoining walls, and the spaces bridged with plate-iron cantilever beams. The result, as verified by bench-marks, has proved the sufficiency of the piling under the concentrated stress over the voids, which was spaced and driven in accordance with the formula here submitted.

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THE GROINED ARCH AS A COVERING FOR RESERVOIRS AND SAND FILTERS: ITS STRENGTH AND VOLUME.

By LEONARD METCALF, Assoc. M. Am. Soc. C. E.

TO BE PRESENTED SEPTEMBER 6TH, 1899.

Some months ago the author had occasion to figure the volume of masonry in the elliptical groined arch roof covering the sand filter then under construction by William Wheeler, M. Am. Soc. C. E., for the Somersworth, N. H., Water-Works (see Plate VI, Fig. 1), and deduced therefor a series of formulas giving the volume of masonry in its cylindrical, cloistered and groined elliptical arches. These formulas, which have been considerably amplified, and to which have been added a sketch of the development of the groined arch, a comparison of the relative volume of masonry in the several types of arches, and a brief description of a method for computing the strength of the groined arch, form the basis of this paper.

The results thus far obtained by the author are presented at this time, not with the idea that they are in any sense complete, but in the hope that they may be of some service to engineers, and that they

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may, perhaps, suggest to others new methods of analysis which may lead to the deduction of general formulas, now entirely wanting, for the dimensioning and comparison of the different types of arches.

Among the most common forms of masonry roofs in use are the following: *First*, the dome or spherical arch; *second*, the cloistered arch; *third*, the cylindrical arch, and *fourth*, the groined arch—all still further subdivided, according to the form of the generating curve into semi-circular, elliptical, segmental, parabolic, pointed or Gothic, multi-centered and various other forms.

Of these four general types of arches, the dome and the cylindrical or barrel arch are too well known to require description. The cloistered and the groined arch, however, should be carefully distinguished, the essential difference being that in the cloistered arch the vaulting springs from a series of springing lines and meets at the crown in a common point, the several vault intersections or arrises forming re-entrant angles in the masonry; whereas, in the groined arch, the vaulting springs from a series of springing points or pillars and meets at the crown in a series of intersecting lines (see Figs. 1 and 2 and photographs of Somersworth, N. H., and Ashland, Wis., filter plants. Plate VI), the several vault intersections or arrises forming so-called groins or ribs, convex angles in the masonry.

It will thus be seen that the groined arch, even more than the dome, gives, naturally, effects of great spaciousness, simplicity and stability of form, well adapted to use in such engineering structures as covered reservoirs, etc., in which economy and stability of construction, and air space for convenience in operation and ventilation, are prime requisites, and in which the presence of supporting pillars is no serious objection.

The history of the groined and the cloistered arch is coeval with the development of the early ecclesiastical structures, and is of remote origin, probably dating back to the time of Roman supremacy; for with the rise of Roman power, we find arched or vaulted types of masonry construction supplanting the post and lintel form of the early Grecians, and passing subsequently through many modifications, tending always toward greater ornamentation and splendor.

Necessity was probably the occasion of the invention of the groined arch, for the earliest custom of the Romans, when building two intersecting arches, was to raise one arch above the other, in order to avoid

their intersection, and the consequent formation of groins, a clumsy though satisfactory method where practicable. But situations soon arose in which this method was impracticable on account of lack of head room, and the groined arch was the inevitable result. Moreover, a common form of arrangement of columns and roof in the early public buildings of the Romans was, according to Choisy, that of a large central vaulted hall or church, flanked on either side by secondary vaulted chambers, which necessitated the use of groined arches.

The origin of the groined arch has thus been traced back to the second or third century of the Christian era, and many striking examples of its magnitude are known. Russell Sturgis, in his "European Architecture," and Auguste Choisy, in his admirable and comprehensive work on "L'Art de Bâtir chez les Romains," describe them at length, citing the great Tepidarium or Baths of Caracalla, built at Rome in 215 A. D., of 82 ft. span; the Baths of Diocletian at Rome, built in 290 A. D. and restored by Michael Angelo in the sixteenth century, and the Basilica of Maxentius and Constantine, of 86 ft. span, portions of which are still standing near the Forum at Rome. Yet, while these massive ancient structures, with all their difficulties of erection and construction, excite our interest and wonder, they could scarcely be cited as models of economical construction.

The first groined arches were characterized by their simplicity, the vaulting—generally semi-circular—being unornamented; the groins merely sharp arrises. Later, they furnished the chief decoration of the edifices in which they were built, and were richly carved and ornamented. With the increasing demand for greater unobstructed space and more striking architectural effects in the large baths and halls, the dimensions of the vaulting grew, as did the difficulty of adequately centering the structure for construction. This, the growing tendency to ornamentation of the groins and the introduction of the Gothic arch, probably suggested the use of ribs to re-enforce the groins or arrises, which ribs thus divided the vaults into sections and permitted the reduction of the thickness of the vaulting between them and the discarding of a portion of the heavy centering.

These ribs were at first very simple in section, being generally broad and thin, with rectangular off-sets, much as the architrave of a door; and, even up to the twelfth century, according to Gwilt, "were seldom moulded with more than a simple torus or some fillets." The

FIG. 1.

FIG. 2.

ribs sprung from the square piers supporting the four corners of the vaulted roof, the groins corresponding to the diagonals of the vault; but in the thirteenth, fourteenth and fifteenth centuries, the section of the supporting columns, the form of groining and number of ribs, grew more complex, and in following the changes and multiplication of the number of ribs we may trace the evolution of the fan-shaped, the star and the many other forms of groining developed during those centuries, described at length and fully illustrated in Dr. G. U. Breymann's "Bau Constructions Lehre," page 151; in Auguste Choisy's "L'Art de Bâtir chez les Romains,"* and in Ware's "Tracts on Vaults and Bridges."

It might be noted, in concluding this brief outline of the early development of the groined arch, that the materials for the vaulting used by the Romans were stone or brick, backed with concrete or rubble, and the ribs were generally built of the same material, sometimes specially moulded, the rib joints, particularly in the smaller ribs, being laid either in cement mortar or with sheet lead, to distribute the pressure more uniformly over the joint. In later days wood and iron came to be used in place of masonry as the building material.

Turning now to a consideration of the adaptability of the groined arch as a roof covering for large reservoirs, sand filter beds or similar structures in which large spans are not required nor piers objectionable, the following points of advantage may be claimed for it in comparison with the cylindrical, the cloistered and the spherical or domed arch.

First.—Greater air space or spaciousness under the vaulting, which tends toward better circulation of air and consequent aeration of the water, and which, in the case of sand filter beds, gives more room for scraping and renewing the surface of the beds.

Second.—Economy of material.

Third.—Absence of necessity for lintel arches.

Fourth.—Equal or greater adaptability to lighting, ventilation and access from above.

Fifth.—Ease of construction and equal adaptability to concrete construction, except as compared with the cylindrical arch, for which

* Illustrations from Choisy's work are also to be found in a short paper on "Roman Construction," by G W. Perry, published, since the preparation of this paper, in the *Journal* of the Association of Engineering Societies, October, 1898, Vol. xxi, No. 4.

the centers can be somewhat more easily and cheaply built, though the necessity for centering for the lintel arches in the latter would perhaps off-set this advantage of the cylindrical arch.

The groined arch is, of course, best adapted to roofing square or rectangular areas, for though circular areas can be covered in this way, the greater difficulty of forming the arch centers for the latter is apparent, as they have to be cut around the perimeter of the chamber on a spiral or other curve formed by the intersection of the roofing and the side walls.

The superiority of the groined arch, as compared with other types of arches in the above-mentioned respects, needs no demonstration, except as to the economy of material, which involves two considerations: First, the method of computing the amount of masonry in any system of vaulting; and second, the strength of that vaulting, which will now be discussed.

Assume, for purposes of illustration, a given square or rectangular filter-area which is to be roofed over with a system of groined arches, as shown in Fig. 3, built of concrete, of which the amount is to be computed. It will be noticed that this roof is made up, first, of a series of groined arch units $D\,E\,F\,G$ (or $N\,O\,Q\,R$, which is equivalent to it), of span equal to the distance in the clear between piers, and each supported by one pier, as $H\,I\,J\,K$; second, of a cloistered arch of span $2\,B\,C$ and $2\,A\,B$, made up of the four corners of the area, each corresponding to $A\,B\,C\,D$, joined together, and third, of half of a cylindrical arch extending around the entire area, springing from the retaining wall and ending at the crown of the vault between said retaining wall and the first line of piers, of length $2\,C\,W$ and $2\,A\,V$, respectively. The roof vaulting thus springs from and is supported by the side walls and piers, and forms crown lines parallel to and lying between the several rows of piers, and the piers and side walls, in both directions.

Let us take up, therefore, in turn, the computation of these units —the groined arch unit, the cloistered arch unit, and the cylindrical arch unit.

In considering the easiest method of computing the masonry in these units it has seemed to the author—in view of the different materials of construction used, whether one or several kinds together, as brick or concrete, or both, and the diverse forms of extrados used,

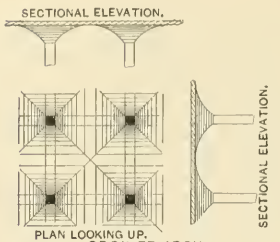


FIG. 1.

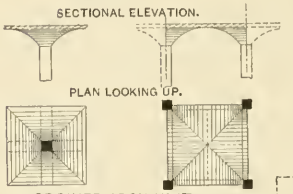


FIG. 4. FIG. 5.

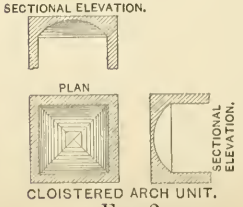


FIG. 2.

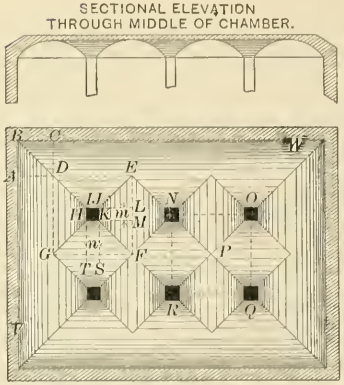


FIG. 3.

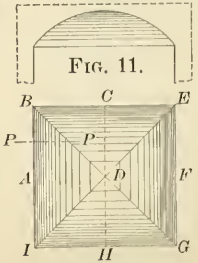


FIG. 11.

FIG. 12.

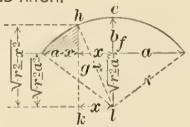


FIG. 13.

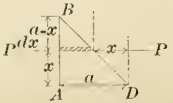


FIG. 14.

FIG. 6.

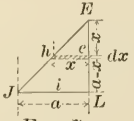


FIG. 7.

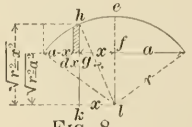


FIG. 8.

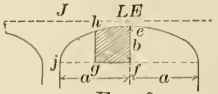


FIG. 9.

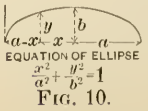


FIG. 10.

often regardless of the materials of construction, according to which the extrados may conform to the soffit or be parallel to it, may be a horizontal plane surface passing through the crown, an inverted pyramid over the piers, or any other surface suiting the particular circumstances or the preference of the designer—that the simplest method was to compute the volume of masonry as that of a circumscribed prism from which has been deducted the volume of the air space between it (said circumscribed prism) and the enclosed masonry. This method has, therefore, been adopted, and in Table No. 1 will be found the values now to be deduced for the volume of the air space between the horizontal plane of the springing lines and the soffit of various unit-types of vaulting, by the aid of which, in the manner above described, the volume of masonry may be computed.

The Groined Arch Unit.

The groined arch unit of masonry may be defined, for the purposes of this discussion, as the masonry in a system of groined arch vaulting supported by one pier and contained within the four vertical planes passing through the central axes and crowns of the arches springing therefrom. This unit, shown in Fig. 4, is equivalent to the unit (Fig. 5) contained within four vertical planes passing through the centers of four adjacent piers, and becomes identical with it if the several parts of the latter be but rearranged.

The Segmental Groined Arch Unit.—Let us consider first the general case of the (circular) segmental groined arch unit—the semi-circular groined arch unit being but the particular case in which the rise of the arch is equal to half the span—and

Let a = half the span of the arch;
b = rise of the arch;
r = radius of the arch;
d = diameter of semi-circular arch;
m = length of supporting piers;
n = width of supporting pier;
w = width or length of pier if the latter be square;
t = thickness of masonry at crown.

Referring to Fig. 4, the volume of air space between the plane of the springing lines of the soffit and of the vaulting is equal to eight times the volume of the "$\frac{1}{8}$-unit" air space, so-called [being that seg-

ment of a cylinder underlying the portion of the vaulting shown in plan by JEL (Fig. 3), enclosed by a horizontal plane passing through the springing line, by the soffit of the arch and by three intersecting vertical planes passing through JE, EL and LJ], plus the volume of a segmental cylindrical arch of span $2a$, rise b, and length equal to $(m + n)$, the width plus the length of the pier.

The volume of the $\frac{1}{8}$-unit air space may be found in the following manner: Passing a vertical plane, perpendicular to EL, through any point distant x from E in the line EL, it will cut from the $\frac{1}{8}$-unit air space a quadrilateral $efgh$, shown in Figs. 6 and 7, of which the line eh is the segment of a circle, and the area of which is indicated by the expression:

$$\int_o^x \left(\sqrt{r^2 - x^2} - \sqrt{r^2 - a^2}\right) dx \quad \ldots\ldots (1)$$

$$= \frac{x}{2}\sqrt{r^2 - x^2} + \frac{r^2}{2}\sin^{-1}\frac{x}{r} - x\sqrt{r^2 - a^2} \quad \ldots\ldots (2)$$

as may be readily deduced from Figs. 6, 7 and 8.

The volume of air space underlying a $\frac{1}{8}$-unit of the (circular) segmental groined arch is therefore found by integrating the above expression (2) between the limits of a and 0, thus:

$$\int_o^a \left(\frac{x}{2}\sqrt{r^2 - x^2} + \frac{r^2}{2}\sin^{-1}\frac{x}{r} - x\sqrt{r^2 - a^2}\right) dx$$

$$= \frac{(r^2 - a^2)^{\frac{3}{2}}}{3} + \frac{ar^2}{2}\sin^{-1}\frac{a}{r} - \frac{r^3}{3} \quad \ldots\ldots (3)$$

$$= \frac{(r - b)^3}{3} - \frac{r^3}{3} + \frac{ar^2}{2}\sin^{-1}\frac{a}{r} \quad \ldots\ldots (4)$$

The volume of the segmental cylindrical arch, of span $2a$, rise b, and length $(m + n)$ is derived similarly to (1), and is indicated by the expression:

$$(m + n)\int_{-a}^{a} \left(\sqrt{r^2 - x^2} - \sqrt{r^2 - a^2}\right) dx$$

$$= (m + n)\left(r^2 \sin^{-1}\frac{a}{r} - a\sqrt{r^2 - a^2}\right) \quad \ldots\ldots (5)$$

or

$$= (m + n)\left[r^2 \sin^{-1}\frac{a}{r} - a(r - b)\right] \quad \ldots\ldots (6)$$

hence, the total volume of air space of the (circular) segmental groined arch unit between the plane of springing lines and soffit of arch is equal to $8 \times$ (4) plus (5) or (6).

$$= \frac{8}{3}(r - b)^3 - \frac{8}{3}r^3 + (4a + m + n)r^2 \sin^{-1}\frac{a}{r} - a(m + n)(r - b) \quad \ldots\ldots (7)$$

If the pier is square and $m = n = w$, (7) becomes,

$$\frac{8}{3}(r-b)^3 - \frac{8r^3}{3} + 2r^2(2a+w)\sin^{-1}\frac{a}{r} - 2awr(r-b) \quad (8)$$

If the pier area becomes equal to 0, *i. e.*, a point, (7) becomes,

$$\frac{8}{3}(r-b)^3 - \frac{8r^3}{3} + 4ar^2\sin^{-1}\frac{a}{r} \quad (9)$$

With the aid of the above formulas the volume of the (circular) segmental groined arch unit may readily be found when the radius, span, rise and crown thickness of arch, dimensions of piers and form of extrados are known. Thus, using the same nomenclature as before (see page 296), if the spandrel of the arches be filled with masonry so that the extrados is a horizontal plane surface passing through the crown, at which the depth of masonry is equal to t, the volume of masonry in the (circular) segmental groined arch unit becomes,

$$(2a+m)(2a+n)(b+t) - \text{formula (7)}$$

$$= (2a+m)(2a+n)(b+t) - \frac{8}{3}(r-b)^3 + \frac{8}{3}r^3 - (4a+m+n)$$

$$r^2\sin^{-1}\frac{a}{r} + a(r-b)(m+n) \quad (10)$$

or if the pier is square and $m = n = w$,

$$(2a+w)^2(b+t) - \frac{8}{3}(r-b)^3 + \frac{8}{3}r^3 - 2r^2(2a+w)\sin^{-1}$$

$$\frac{a}{r} + 2awr(r-b) \quad (11)$$

or if the pier area is 0, *i. e.*, a point,

$$4a^2(b+t) - \frac{8}{3}(r-b)^3 + \frac{8}{3}r^3 - 4ar^2\sin^{-1}\frac{a}{r} \quad (12)$$

If the extrados filling be depressed over the piers in the form of an inverted pyramid, the volume of masonry would be found by deducting from (10), (11) or (12) the volume of that inverted pyramid.

Or, finally, if the extrados also is segmental in form, *i. e.*, is parallel to the soffit of the vaulting, the volume of masonry in the (circular) segmental groined arch unit would be found by deducting the volume between the plane of the springing lines and the soffit of the vaulting, as determined by formulas (7), (8) or (9), from the volume between the plane of the springing lines and the extrados of the vaulting as determined by formulas (7), (8) or (9), substituting therein, for the values of the radius, span and rise of the soffit arch, those functions of the extrados arch, and, for the dimensions of piers, the new values determined from the section cut by the extrados from the pier at the plane of the springing lines.

Semi-Circular Groined Arch Unit.—Similar values for the semi-circular groined arch unit may be obtained directly by substituting in the formulas just deduced for the (circular) segmental groined arch the value of the semi-span a and rise b of the arch, their equivalent, the radius of the arch r or half the diameter, $\frac{d}{2}$, and are to be found in Table No. 1.

The Elliptical Groined Arch Unit.—In deducing general formulas for the computation of the masonry in the elliptical groined arch unit of vaulting, the same general methods of analysis can be used as were followed in the case of the (circular) segmental groined arch vaulting, the only difference being that the transverse curve of the arch varies according to the law of the ellipse, $\frac{x^2}{a^2} + \frac{y^2}{b^2} = 1$, instead of that of the circle $y = \sqrt{r^2 - x^2} - \sqrt{r^2 - a^2}$.

Referring to Figs. 9 and 7, the volume of air space of the $\frac{1}{8}$-unit elliptical groined arch between the plane of the springing lines and the soffit of the arch is

$$\frac{b}{a}\int_0^a x\sqrt{2ax - x^2}\ dx$$
$$= 0.4521\ a^2 b \ldots\ldots\ldots\

crown (at which point the masonry has a depth or thickness equal to t) is, when the pier area is $m \times n$.

$$(2a + m)\ (2a + n)\ (b + t) - 3.6168\, a^2\, b - 1.5708\, a\, b\, (m + n) \ldots (26)$$

When the pier is square,

$$= (2a + w)^2\ (b + t) - 3.6168\, a^2\, b - \pi\, a\, b\, w \ldots\ldots\ldots\ldots (27)$$

When the pier area is zero,

$$= 4\, a^2\, t + 0.3832\, a^2\, b \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Any difference in the form of the extrados or back of the vaulting may be treated in a manner precisely similar to that suggested for the segmental groined arch vaulting.

Elliptical Segmental Groined Arch Unit.—Formulas have not been deduced for this form of groined vaulting on account of the improbability of its being used; for if a segmental arch is required, the circular form would probably be used in preference to the elliptical. In any event, the deduction of formulas to cover this case would be similar to that already outlined for the circular segmental groined arch.

Parabolic Groined Arch Unit.—The volume of masonry involved in the parabolic groined arch unit—that is, a groined arch of which the right section through the crown is a parabola with vertical axis and vertex at the crown—may be computed in the same way as the circular and elliptical groined arch units; bearing in mind that the equation of the curve has to be modified to fit the parabola, $x^2 = \frac{a^2\, y}{b}$.

The parabolic groined arch will be found most convenient for use as an invert for the floor of the reservoir or filter bed, on account of its similarity to the segmental groined arch, and the ease with which grade stakes can be set for it.

Referring to Figs. 9 and 7, the volume of air space of the $\frac{1}{8}$-unit parabolic groined arch between the plane of the springing lines and the soffit of the arch is:

$$= \frac{5\, a^2\, b}{12} \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

The volume of the parabolic cylinder, of span $2\,a$, rise b, and length $m + n$, is

$$= \frac{4\, a\, b}{3}\ (m + n) \ldots\ldots\ldots\ldots\ldots\ldots (30)$$

Hence, the total volume of air space between the plane of the springing lines and the soffit of the vaulting of a parabolic groined arch unit is, when the pier area equals $m \times n$,

$$\frac{10\, a^2\, b}{3} + \frac{4\, a\, b}{3}\ (m + n) \ldots\ldots\ldots\ldots\ldots\ldots (31)$$

When the pier area is square,

$$= \frac{10\,a^2\,b}{3} + \frac{8\,a\,b\,w}{3} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (32)$$

When the pier area is zero,

$$= \frac{10\,a^2\,b}{3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

from which the volume of masonry may be computed as previously outlined.

Cloistered Arch Unit.

Let us now deduce formulas covering the same ground for the cloistered arch; the second step (see page 294) in the computation of the masonry of the hypothetical roof under discussion.

The cloistered arch unit differs from the groined arch unit, in that, whereas the groined arches spring from piers in points (or lines of length equal to the dimensions of the pier), and meet at the crown in intersecting lines of lengths equal to the span plus the width and length, respectively, of the pier, the cloistered arches spring from the side walls, or lintel arches, in lines circumscribing the unit, and intersect at the crown in a single point. Hence the same general method of computation and the same mathematical forms as were used for computing the masonry in groined vaulting may be used for cloistered vaulting, if only the proper limits for integration be substituted therein.

Segmental Cloistered Arch Unit.—Adopting the same nomenclature as before, we find in determining the volume of air space in the $\frac{1}{8}$-unit cloistered arch shown in plan by $A\ B\ D$, Figs. 12 and 14, that the vertical cutting plane $P\ P$, drawn perpendicular to $A\ B$ at a distance x from A, intersects therefrom a partial segment $g\,h\,j$, whose area,

if a = semi-span,

r = radius of curve of soffit of segmental cloistered arch,

$b = r - \sqrt{r^2 - a^2}$ = rise,

is equal to,

$$\int_x^a (\sqrt{r^2 - x^2} - \sqrt{r^2 - a^2})\,dx$$

$$= \frac{r^2}{2}\sin^{-1}\frac{a}{r} - \frac{r^2}{2}\sin^{-1}\frac{x}{r} + \frac{(2\,x - a)}{2}\sqrt{r^2 - a^2}$$

$$- \frac{x}{2}\sqrt{r^2 - x^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)$$

Hence the volume of air space between the plane of the springing lines and the soffit of the arch of the $\frac{1}{8}$-unit segmental cloistered arch is

$$\int_o^a [\text{Formula (34)}]\, dx$$

$$= \frac{(r^2 - a^2)^{\frac{3}{2}}}{6} - \frac{r^2}{2}\sqrt{r^2 - a^2} + \frac{r^3}{3} \quad \ldots\ldots (35)$$

$$= \frac{b^2}{6}(3r - b) \quad \ldots\ldots (36)$$

And the volume of air space between the plane of the springing lines and the soffit of the vaulting of a segmental groined arch unit, of span $2a$, and rise b, is

$$= \frac{4b^2}{3}(3r - b) \quad \ldots\ldots (37)$$

Semi-Circular Cloistered Arch Unit.—By making the rise (which is equal to the semi-span) equal to the radius of the arch in formulas (34), (35), (36) and (37), for the segmental cloistered vaulting, we have corresponding formulas for the semi-circular cloistered arch (see Table No. 1)

Elliptical Cloistered Arch Unit.—Similarly, the volume of the air space under the $\frac{1}{8}$-unit elliptical cloistered arch is

$$\int_o^a \frac{b}{a}\left[\frac{(x-a)}{2}\sqrt{2ax - x^2} + \frac{a^2}{2}\,\text{vers}^{-1}\frac{x}{a}\right] dx \quad \ldots\ldots (38)$$

$$= \frac{a^2 b}{3} \quad \ldots\ldots (39)$$

And the volume of the air space under the elliptical cloistered arch unit is,

$$\frac{8a^2 b}{3} \quad \ldots\ldots (40)$$

Parabolic Cloistered Arch Unit.—The volume of the air space under the $\frac{1}{8}$-unit parbolic cloistered arch is,

$$= \int_o^a \left[\frac{2(a\ b)}{3} - b\ x + \frac{(b x^3)}{3a^2}\right] dx \quad \ldots\ldots (41)$$

$$= \frac{a^2 b}{4} \quad \ldots\ldots (42)$$

And the air space under the parabolic cloistered arch unit is

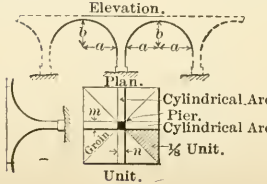
$$2a^2 b \quad \ldots\ldots (43)$$

By the aid of these formulas may be found, as previously described, the volume of masonry in any cloistered arch, by deducting from the computed volume of a circumscribed prism the volume of the air space underlying the soffit of the vaulting, together with the volume of the air space, if any, between the extrados and the horizontal upper base of the prism which passes through the crown. It

TABLE No.1. Volume of Air-Space between Soffit of Arch & Plane through Springing Lines.

Type of Arch		Segmental	Semi-Circular	Elliptical	Parabolic
Groined Arch.	Unit of Pier Area $m \times n$	$=\frac{8}{3}(r-b)^3-\frac{8}{3}r^3+(4a+m+n)r^2\sin^{-1}\frac{a}{r}-a(m+n)(r-b)$	$=\pi r^2(2r+\frac{m+n}{2})-\frac{8}{3}r^3$. $=\frac{\pi d^2}{4}(d+\frac{m+n}{2})-\frac{d^3}{3}$.	$=\frac{\pi ab}{2}(m+n)+8a^2b(0.4521)$ $=1.5708ab(m+n)+3.6168\,a^2b$	$=\frac{10}{3}a^2b+\frac{4}{3}\frac{ab(m+n)}{(m+n)}$ $=3.333\,a^2b+1.333\,ab$
	Pier Area Square	$=\frac{8}{3}(r-b)^3-\frac{8}{3}r^3+2r^2(2a+w)\sin^{-1}\frac{a}{r}-2aw(r-b)$.	$=\frac{\pi d^2}{4}(d+w)-\frac{d^3}{3}$.	$=3.1416\,abw+3.6168\,a^2b$.	$=\frac{10}{3}a^2b+\frac{8}{3}abw$.
	Pier Area Zero	$=\frac{8}{3}(r-b)^3-\frac{8}{3}r^3+4ar^2\sin^{-1}\frac{a}{r}$.	$=8r^3(\frac{\pi}{4}-\frac{1}{3})=3.6168r^3$.	$=3.6168\,a^2b$	$=\frac{10}{3}a^2b=3.3333\,a^2b$.
	⅛-Unit.	$=\frac{(r^2-a^2)^{\frac{3}{2}}}{3}+\frac{ar^2}{2}\sin^{-1}\frac{a}{r}-\frac{r^3}{3}$ $=\frac{(r-b)^3}{3}-\frac{r^3}{3}+\frac{ar^2}{2}\sin^{-1}\frac{a}{r}$.	$=r^3(\frac{\pi}{4}-\frac{1}{3})$ $=0.4521r^3$.	$=a^2b(\frac{\pi}{4}-\frac{1}{3})$ $=0.4521\,a^2b$.	$=\frac{5}{12}a^2b$. $=0.4166\,a^2b$.
Cloistered Arch.	Unit.	$=\frac{4}{3}b^2(3r-b)$.	$=\frac{8r^3}{3}=2.6667r^3$.	$=\frac{8}{3}a^2b=2.6667\,a^2b$.	$=2\,a^2b$.
	⅛-Unit.	$=\frac{b^2}{6}(3r-b)$.	$=\frac{r^3}{3}=0.3333\,r^3$.	$=\frac{a^2b}{3}=0.3333\,a^2b$.	$=\frac{a^2b}{4}=0.25\,a^2b$.
Cylindrical Arch.		$=l\left[r^2\sin^{-1}\frac{a}{r}-a(r-b)\right]$	$=\frac{\pi a^2 l}{2}=\frac{\pi r^2 l}{2}$.	$=\frac{\pi abl}{2}$.	$=\frac{4}{3}\frac{abl}{3}=1.3333\,abl$.
Rectangular Prism		$=2\,abl=4\,a^2b$ *when* $l=2a$ $=(2a+m)(2a+n)(b+t)$	←	←	←

References:



a = semi-span of arch.
b = rise of arch.
r = radius of arch.
d = diameter of semi-circular arch.
m = length of supporting pier.
n = width of supporting pier.
w = length of square pier.
l = length of cylindrical arch.
t = thickness of concrete at crown.

Note: Air-space "unit" = space underlying portion of roof supported by one pier and contained within the four vertical planes passing through the central axes and crowns of the arches springing therefrom.

should be noted, moreover, that the dimensions (length and width) of the prism circumscribing the cloistered arch are not merely equal to the clear span in both directions, but are in each case greater than those spans by an amount equal to the combined thickness of the vaulting at the two opposite springing lines, and hence equal to the span plus twice the thickness of the abutments.

Cylindrical Arch Units.

Formulas for the volume of the air space between the plane of the springing lines and the soffit of the cylindrical arch units have already been deduced, but are gathered here for convenience:

For the segmental cylindrical arch,

$$= l\left[r^2 \sin^{-1}\frac{a}{r} - a\,(r - b)\right] \quad \ldots\ldots(44)$$

For the semi-circular cylindrical arch,

$$= \frac{\pi r^2 l}{2} \quad \ldots\ldots(45)$$

For the semi-elliptical cylindrical arch,

$$= \frac{\pi a b l}{2} \quad \ldots\ldots(46)$$

For the parabolic cylindrical arch,

$$= \frac{4 a b l}{3} \quad \ldots\ldots(47)$$

in all of which

l = length of cylindrical arch,
r = radius of arch,
a = semi-span of arch,
b = rise of arch.

Tables.

Table No. 1 contains, for convenience in computation, the formulas just deduced relating to the volume of the air space between the plane of the springing line and the soffit of the vaulting of the different types of vaulting discussed.

In Table No. 2 is given a comparison of the ratios of the volume of masonry in the groined, cloistered, cylindrical, and dome arch units, the pier area and width of abutment of which is equal to 0, and of which the extrados is a plane surface passing through the soffit crown to the volume of masonry in a circumscribed rectangular prism, passing through the same springing lines and soffit crown; which

table, though misleading in certain respects, has, perhaps, a comparative value, and is to that extent suggestive.

Table No. 3 contains a comparison of the amounts of masonry in the elliptical and semi-circular groined, cloistered and cylindrical arch units, with that in a circumscribed rectangular prism passing through the springing lines and the (extrados) crown of the vaulting, based upon the following, and, perhaps, more satisfactory assumptions:

Span of arch..... $= 2a$;

Rise of elliptical arches..... $= \frac{1}{4}$ of span $= \frac{a}{2}$;

Piers square;

Width of piers or thickness of abutments..... $= \frac{1}{6}$ to $\frac{1}{24}$ of span;

Thickness of masonry at crown.. $= \frac{1}{32}$ of span;

Spandrels of masonry level with crown (that is, extrados crown).

TABLE No. 2.—Comparison of Masonry in Groined, Cloistered, Cylindrical, and Dome Arch Units; with Volume of Circumscribed Rectangular Prism passing through Springing Lines and Crown (of Soffit, not of Extrados) of Arch Unit.

Pier area and width of abutment = 0, and extrados a horizontal plane surface through soffit crown:

Description of "Unit."	Air space under masonry.	Volume of masonry. Cubic feet.	Ratio of volume of masonry to circumscribed rectangular prism.
Rectangular Prism	.....	$4a^2b$	100%
Elliptical Groined Arch, Area of pier = 0	$3.6168a^2b$	$0.3832a^2b$	9.58%
Circular Groined Arch, Area of pier = 0	$3.6168a^3$	$0.3832a^3$	$9.58\% \times \frac{a}{b}$
Parabolic Groined Arch, Area of pier = 0	$3.3333a^2b$	$0.6666a^2b$	16.66%
Elliptical Cloistered Arch	$2.6667a^2b$	$1.3333a^2b$	33%
Circular Cloistered Arch	$2.6667a^3$	$1.3333a^3$	$33\frac{1}{3}\% \times \frac{a}{b}$
Elliptical Cylindrical Arch ..... $l = 2a$..	$3.1416a^2b$	$0.8584a^2b$	21.46%
Circular Cylindrical Arch..... $l = 2a$..	$3.1416a^3$	$0.8584a^3$	$21.46\% \times \frac{a}{b}$
Hemispherical Dome, of equivalent area .... $R = 1.1284a$	$2.0944R^3$	$1.0472R^3$	37.61%

References:
a = semi-span of arch.
b = rise of arch.
R = radius of arch.
L = length of cylindrical arch.

TABLE No. 3.—Comparison of Masonry in Groined, Cloistered and Cylindrical Arch Units, with Volume of Circumscribed Rectangular Prism, based upon the Following Assumptions: Span of Arch $= 2a$; Rise of Elliptical Arches $= \frac{1}{4}$ of Span; Piers, Square; Width of Piers or Abutment $= \frac{1}{6}$ to $\frac{1}{24}$ of Span; Thickness of Concrete at Crown $= \frac{1}{32}$ of Span; Spandrels filled with Masonry Level with Crown (of Extrados).

Description of arch.	Volume of masonry.	Volume of circumscribed prism.	Ratio %
Elliptical Groined Arch. Pier thickness $= \frac{1}{8}$ span	$0.729\,a^3$	$3.0625\,a^3$	23.8
Semi-circular Groined Arch. Pier thickness $= \frac{1}{6}$ span	$1.121\,a^3$	$5.785\,a^3$	21.0
Parabolic Groined Arch	$0.9514\,a^3$	$3.0625\,a^3$	31.6
Elliptical Cloistered Arch, with abutment thickness $= \frac{1}{12}$ span	$0.7292\,a^3$	$3.0625\,a^3$	23.8
Elliptical Cloistered Arch, with abutment thickness $= \frac{1}{8}$ span	$1.3333\,a^3$	$4.0000\,a^3$	33.3
Circular Cloistered Arch, with abutment thickness $= \frac{1}{12}$ span	$3.1180\,a^3$	$5.7847\,a^3$	53.9
Circular Cloistered Arch, with abutment thickness $= \frac{1}{8}$ span	$4.8889\,a^3$	$7.5556\,a^3$	64.7
Circular Cylindrical Arch, with abutment thickness $= \frac{1}{12}$ span	$2.1195\,a^3$	$5.7847\,a^3$	36.7
Circular Cylindrical Arch, with abutment thickness $= \frac{1}{24}$ span	$1.5844\,a^3$	$4.9878\,a^3$	31.8
Elliptical Cylindrical Arch, with abutment thickness $= \frac{1}{12}$ span	$1.2299\,a^3$	$3.0625\,a^3$	40.2
Elliptical Cylindrical Arch, with abutment thickness $= \frac{1}{24}$ span	$0.9389\,a^3$	$2.6406\,a^3$	35.6

The above comparisons are not absolute, as the dimensions of the circumscribing prism vary. Moreover, no allowance has been made in them for the masonry required for the lintel arches from which spring the cloistered and cylindrical arches.

The results are certainly interesting, and while the author does not propose to discuss them here—as they speak for themselves—he would call attention to the fact that they are not strictly comparable for a large structure, for the reason that while the span, rise and crown thickness are the same in all cases, the depth of abutment or piers (and consequently the amount of masonry), varies, and for the further reason that no allowance has been made for the amount of masonry in the lintel arches required by the cloistered and cylindrical arches. This latter circumstance, however, is to the advantage of the groined arch in a consideration of its economy, as compared with other types of vaulting.

Method of Computing Strength of Arch.

Turning now from the subject of the volume of masonry in the groined arch, let us briefly touch upon a method for studying or computing the stability of the groined arch.

Judging from the literature on the subject, but little study has been given to the stability of the groined arch by mathematicians and engineers in bygone years. Dr. Hermann Scheffler, in his "Theorie der Gewölbe Futtermauern und Eisernen Brücken," published in 1857, presented perhaps, at least so far as the author has observed, the most complete analysis of the subject that has yet been published, but it is based, like his discussion of the cylindrical arch, on the assumption of the incompressibility of the building material. Rondelet, at about the same time, carried on a series of experiments with different types of arches upon small models, which experiments, though they seemed to bear out Scheffler's deductions, can hardly be accepted as more applicable to full-sized arches than are the early experiments of Hodgkinson and Fairbairn upon small models applicable to full-sized specimens of timber, iron and steel.

William Cain, M. Am. Soc. C. E., has also published a short article* on "Voussoir Arches Applied to Stone Bridges, Tunnels, Domes and Groined Arches" which will be found helpful, and which was written in continuation or amplification of his "Theory of Voussoir Arches."†

More recently, however, De Grande and Résal,‡ treating of "Groined Arches," have outlined a method which appears to the author to be rational at least, if not so easy of application as an empirical formula, and which is herewith briefly abstracted and translated:

"* * * Let us consider, then, the conditions of stability which a work of this character presents.

"Let us suppose that we cut one of the two vaults by two vertical planes very close together, and normal to the generatrices $F\,G$ and $K\,L$, Fig. 255. The portion of the arch thus cut out is a cylindrical vault of infinitesimal length, and has for its right section, symmetrical with reference to the vertical plane through the crown, a fraction of the section of the complete cylinder $A\,B$ limited by the vertical planes $K\,F$ and $L\,G$. We can compute the thrust exerted by this arch on its abutments, and determine the point of intersection V of the line of resistance, in the plane of the section. Let P be the weight of this half incomplete arch and Q the thrust which it exerts.

"Let us consider the portion $L'\,G'\,K\,F$ of the arch which corresponds exactly to that previously considered on the adjacent vault.

* No. 42, Van Nostrand Science Series, 1879.

† No. 12, Van Nostrand Science Series.

‡ "Encyclopédie des Travaux Publics," Ponts en maçonnerie, par De Grande et J. Résal. Vol. 1, page 329.

Considering the identity which exists between these two fractions of the arch from the point of view of the load, the length and the right section, the last will exert the same horizontal thrust Q, and its line of resistance will cut the vertical plane through $A\ C$ at the same point V.

"The lines of resistance of these two arches therefore meet in the same plane of the groin; the vertical components P of these pressures combine, the one with the other, and the horizontal components, that is to say, the thrusts which cut each other at right angles have for their resultant a horizontal force equal to $Q\sqrt{2}$, and contained in the vertical plane $A\ C$.

"Resuming now, the load supported by the portion of the groined arch $L\ G$, $F\ K$, $L'\ G'$ has for a result the development, in the plane of the groin $A\ C$, of a force of vertical component $2\ P$, and horizontal component $Q\sqrt{2}$, and applied at a point V, which we know how to find.

"Let us suppose that we divide the half of the vaulted arch $A\ B\ S\ D$ by a series of vertical planes, such as $F\ G$, $K\ L$, $F\ G'$, $K\ L'$, into a certain number of slices, and that we determine by the preceding method, in amount and direction, the resultant pressures transmitted by these slices in the plane of the groin $A\ S$.

"Fig. 256 represents the vertical section of the groined arch cut by the plane $A\ S$, and we have marked thereon the points of application $V_1\ V_2\ V_3$, * * * V_n of the resultants corresponding to the successive slices cut from the arch. It will suffice, to combine these different forces acting from the crown S toward the springing A, to obtain the curve of the line of resistance in the plane of the groin.

"The total weight ΣP transmitted to the upper section of the column will be equal to the weight of a quarter of the groined arch $A\ S\ B$, and the total thrust will be the sum of the partial thrusts calculated separately for the different slices.

"Let us cut the groin $A\ S$ by a plane $M\ N$, Fig. 255, which is perpendicular to it. Let W (Fig. 257) be the point of intersection in this plane of the line of resistance, and T the resultant of the reactions transmitted by the portion of the groined arch which lies on the groin between the crown S and the plane $M\ N$. This force T will distribute itself over a certain zone of the masonry, over one portion and another of the plane of symmetry $A\ S$, and as the total reaction of the arch will be definitely transmitted to the column A, it is natural to suppose that the zone of masonry affected by the force T will be sensibly limited by the vertical planes $R\ S$ and $T\ S$ limited by the crown and the opposite corners T and R of the column. This statement seems to us to be almost self-evident.

"Fig. 258 represents the section cut by the plane $M\ N$ from this prism of masonry which is called upon to balance by its molecular

action the force T. In order that the work may be stable it is necessary that the maximum pressure developed in this section should not exceed the practical limit; we can also calculate under all circumstances the value of this maximum pressure by the formulas for the resistance of materials.

"Three cases may present themselves: First, when the calculated pressure is admissible. The work will then have the desired stability without modification. Second, when the point W is too near the upper summit M, corresponding to the re-entrant angles of the profile. It is necessary then to reinforce the top of the section of masonry, and one can do this without difficulty (Fig. 258) by filling in the depression in the surface of the extrados of the groined arch in the plane $A\,S$. This reinforcement of the theoretical diedral angle by a regular surface, or one bringing the extradoses of the two vaults into accord, is very frequently practiced by builders. Third, when the point W is too near the inferior summit N corresponding to the groined edge of the work. This groin runs the danger, then, of failing and of crushing, in consequence of excessive compression. It is therefore necessary to reinforce it. This can be done by surrounding the groin with a prism of masonry forming a rib and intimately bonded to the mass of the groined arch (Fig. 259).

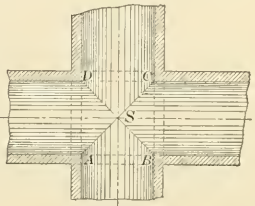
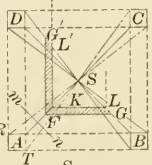


FIG. 253.

FIG. 255.



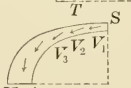


FIG. 256.

FIG. 258.

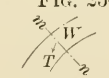


FIG. 257.

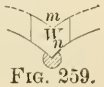


FIG. 259.

"This is a custom well known to architects, which is entirely justified, as we see, by the theory of arches. Sometimes this rib stops at its junction with the column, on the vertical ornament upon which it rests; or, again, it may continue from its junction with the column to the bottom of the latter, following the vertical edge A. Fig. 260 represents the plan of a work thus designed, and it is customary to place at the crown a salient stone on which the ribs of the groin S abut, and which forms a motive of decoration. * * * It happens sometimes that groined arches are formed by the intersection of two vaults of the same rise but different span. In this case the thrusts of their vaults being un-

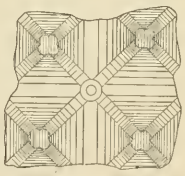


FIG. 260.

equal, the resulting pressures developed in their vertical plane of intersection will no longer necessarily be contained in this plane.

"The above-described method of analysis is perfectly applicable, however, though one recognizes in general that this disposition is less favorable to stability, and should be discarded when arches of large dimensions and heavy loads are concerned."

It is apparent, of course, that this method makes no allowance for the greater stability of the structure due to the monolithic character of the masonry when constructed of concrete; which stability is no doubt materially increased thereby.

Scheffler drew the following conclusions from his analysis of the stability of groined arches, as is to be found in his "Theorie der Gewölbe," page 183:

"It is a striking fact that, by comparison of these different values for E (the thickness of abutment), in all cases the thickness of abutment for a free standing pillar the value of which is computed according to (17), upon which two rib arches act at right angles simultaneously, is smaller than for a simple cylindrical arch, or for the unbalanced pressure resulting from a single arch rib. The reason for this is to be found in the fact that whereas in the first case, where all the forces unite along the diagonal, the moment of the abutment grows according to the $\sqrt{2} = 1.414$ times; under the same conditions, the moment of the injurious (harmful) horizontal forces, only increases at their junction; while the lever arm, therefore, also the moment of the vertical forces favorable to stability, increases in greater proportion, namely, as $\frac{3}{2}\sqrt{2} = 2.121$ times. * * *

"These results (which vary materially for greater coefficients of stability, as we have seen above) find a noteworthy confirmation in the practical investigations with models by Rondelet, which are described in his "L'Art de Bâtir," Book IX, 6th section, chapter 2 (page 327 of the German translation), and according to which, for the limit of equilibrium of equal span widths, the thickness of abutment of a dome arch, of a cloistered arch, of a cylindrical arch and of a groined arch, are about in the proportion of the numbers 1, 3, 4, 6."

Albert Hébrard in his book "L'Architecture" gives an empirical formula for the determination of the thickness of a supporting pier of a groined arch which reduces to the form: "$x = \sqrt{\frac{2\,F\,r\,L}{l}}$ when H is equal to or greater than the span" in which

x = thickness or depth of pier;

F = thrust at crown of arch between piers;

L = distance on centers of crown points, *i. e.*, span plus width of pier;

l = width of pier;

r = coefficient of stability, varying from 1 to 2, according to judgment.

But it will be noted that the thrust on the cylindrical arch has first to be determined, and the formula is at best only indirectly applicable.

Recent Examples in the United States.

But few examples of the use of the groined arch in its simplest form as a reservoir or filter bed covering are to be found, either in the United States or abroad. Their introduction for such purpose in this country was due to William Wheeler, M. Am. Soc. C. E., of Boston, who first made use of them in the construction of the covered sand filters designed by him for the Ashland (Wis.) Water Company in 1895 (see Plate VI, Fig. 2). The groined arches there used were, as appears in Mr. Wheeler's paper on the Ashland filters, read before the New England Water-Works Association, March 10th, 1897:

"Elliptical arches of 15.75 ft. span and 3.50 ft. rise. The arch rings are about 5 ins. thick, consisting of two courses of bricks laid flatwise in Portland cement mortar. The spandrels of the arches and the spaces over the piers and adjacent walls are filled and covered with a backing of Portland cement concrete up to the general level of 4 ft. above the spring of the arches, but sloping down to a height of 2 ft. only, above the springing line at the rear of the outside wall."

The arches cover an effective area of about half an acre, in three compartments, and are protected by an earth embankment 2 ft. in depth covering them.

In 1896 groined arches were again used by the same engineer to cover another sand filter, designed by him for the Somersworth (N. H.) Water Works (see Plate VI, Fig. 1). These arches, which cover an effective area of half an acre, were also elliptical, of span 16 ft., rise 4 ft., built of one course of brick, laid on edge, backed with Portland cement concrete up to a horizontal plane; making the depth of masonry at the crown 6 ins., covered with from 2 to 2½ ft. of earth and gravel.

They were also used by Freeman C. Coffin, M. Am. Soc. C. E., in 1897, as a covering for a reservoir designed by him, and built for the Wellesley (Mass.) Water-Works upon Maugus Hill. These groined arches were also elliptical, constructed entirely of Portland cement concrete, of 12-ft. span, 2-ft. rise and with spandrels filled to a depth

giving 6 ins. at crown, covering a circular reservoir 80 ft. in diameter, and supporting an earth and gravel filling 2 ft. deep.

And, finally, Allen Hazen, Assoc. M. Am. Soc. C. E., has made use of them in the new and excellent sand filter plant recently designed by him, and now under construction, for the Albany (N. Y.) Water-Works.

These examples of the groined arch, in engineering structures of such a character, limited as they are in number, are all in the United States that have thus far (February, 1898), come to the knowledge of the author, but it seems highly probable that with the great awakening in questions relating to bacteriological conditions of water supplies, the construction of covered filters and reservoirs will multiply. And for this reason the subject of groined arches as a covering for reservoirs and sand filters has seemed to the author to be pertinent at this time.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

STREET GRADES AND CROSS-SECTIONS IN ASPHALT AND CEMENT.

Discussion.*

By Messrs. HORACE ANDREWS and ROBERT P. WOODS.

HORACE ANDREWS, M. Am. Soc. C. E.—This carefully prepared paper is of interest, and the double curb is an innovation that may be found useful where other means fail. It is much to be regretted that nothing is said either in the paper or in the discussion as to the cost of the improvement. This is certainly a factor of too much importance to be disregarded by a municipal engineer. Concrete curb with gutter attached, similar to that described in the paper, has been laid, under the writer's specifications, for 90 cents per lineal foot. It will be of interest to know whether the double curb cost twice the above amount. Mr. Andrews

The writer agrees with Messrs. Tillson and Hankinson that the second step would better have been omitted at the curb and placed near the fronts of the buildings.

Two steps might even have been used in such a location. The street once improved is apt to retain its cross-section for many years, while it would appear from the photographic views that many of the buildings will be replaced before very long and then the suggested steps in front of them could be done away with.

The relative widths of sidewalks and carriageway are closely connected with the question of economical improvement. It would seem that the 90-ft. streets would have looked better, would have amply ac-

* Continued from April, 1899, *Proceedings*. See February, 1899, *Proceedings* for Paper, by Robert P. Woods, Jun. Am. Soc. C. E., on the subject.

Mr. Andrews. commodated all traffic and would have been more economically improved with 20-ft. sidewalks and a 50-ft. carriage-way, while the 66-ft. streets, with 30-ft. carriage-ways, would certainly accommodate all vehicle traffic in a place the size of Wabash. It would be much cheaper to construct sidewalks than equal areas of asphalt pavement.

The writer agrees with Mr. North as to the superiority of stone over concrete for curbs. The concrete gutter is peculiarly objectionable on account of the running joint at its outer edge; this is quite apt to wear into a rut, a trouble experienced with all such joints where the wheel traffic is considerable.

It is not apparent that an extra pitch, transversely, was needed for the gutter. The general contour of the street might have been maintained, as usual, to the vertical face of the curb.

It seems to the writer that the author and Mr. Tillson are correct in their objections to an absolutely plane surface for the pavement, although they differ from Mr. North in this respect; but quite as serious an evil is the level surface resulting from the use of a circular-arc cross-section, with gutters of equal elevation. Referring to Section No. 12, it is seen that for 7 ft. at the center of the carriage-way the rise is only $\frac{1}{8}$ in. It is practically impossible to avoid depressions along the center of the street where so slight a rise is attempted.

As to the amount of crown to be given a street with gutters of equal elevation, the writer prefers $\frac{1}{72}$ rather than $\frac{1}{100}$ of the width of carriage-way as giving sufficiently flat transverse slopes and as avoiding depressions, while the water is more rapidly brought to the gutters. The ratio $\frac{1}{72}$ is practically that used by the author, the radius of his circular segment then being nine times its chord. The variation between the circle and the parabola is so extremely minute that the cross-section may be termed circular with perfect propriety. The ordinate of the parabola is $y = \frac{x^2}{2r}$, while the ordinate of the circular arc is $y = \frac{x^2}{2r} + \frac{x^4}{8r^3} + \frac{x^6}{16r^5} +$ etc. In the author's example, for the maximum ordinate, $x = 18.67$, $r = 9 \times$ width $= 336.06$, sufficiently near (the author gives $r = 335.30$). For the parabola y will be 0.5186, while the second term, $\frac{x^4}{8r}$, increases this only by 0.0004, the third term being too small to be considered.

The writer, a number of years ago, abandoned the circular-arc form of cross-section and has adopted, for some 20 miles of pavement on concrete foundation, a form intermediate between the circular arc and straight lines forming chords of half the arc.

According to Thomas Codrington, in his article on roads,* this form is used in London, and Mr. Hankinson's proportion of 8 to 5, as that

*Encycl. Britannica.

which the crown bears to the ordinate at the quarter, also agrees with this cross-section. A useful comparison between this form and the circular arc can be made by considering the rates per cent. of the slopes at different parts of the transverse section. If R represents the ratio of width to crown, 72 : 100, etc., and the carriage-way is transversely divided into eighths; then the rates per cent. in the four divisions lying between the center and the gutter, starting at the center, are: For the circular arc, $\frac{50}{R}$, $\frac{150}{R}$, $\frac{250}{R}$ and $\frac{350}{R}$, and for the improved form mentioned, $\frac{125}{R}$, $\frac{175}{R}$, $\frac{225}{R}$ and $\frac{275}{R}$, respectively. For example, with $R = 72$, the slopes of the circular arc are 0.695%, 2.08%, 3.47% and 4.86%, while the preferred cross-section has slopes of 1.74%, 2.43%, 3.13% and 3.82%, being less flat in the center and flatter at the gutters than the circular arc. By thus considering the rates per cent. of different portions of the cross-section, it is easy to perceive the effect of unequal heights of gutters if the slope of the line between them is expressed in the rate per cent. Mr. Andrews.

It would be interesting to know from the author how nearly he succeeded, practically, in securing the cross-sections called for in his theoretical investigations. The writer has found it quite impossible in dense, wet clay to form the subgrade to exact shape. A 10-ton roller will make waves and depressions both longitudinal and transverse, yet the concrete and pavement will stand well when the work is complete. In the writer's practice the curbs are first placed, and, by means of a cord stretched between them and measured offsets the subgrade is approximately formed and is then rolled. The cord is again used and iron rods are driven at the eighths of transverse width; the top of the concrete is marked on these with chalk and the subgrade is then shaved off, generally, or filled up, rarely, until it is exactly the proper distance below the chalk marks. Concrete is then spread and, afterward, the iron rods are pulled up. For a brick pavement fine sand is spread on the concrete, and it is then struck by means of a templ'ate resting on rollers running on the curbs. This template is formed with considerable exactness and can be used on widths of carriage-way up to 36 ft. For greater widths a template of half the width of the carriage-way is used, running on one curb and on a timber laid temporarily in the center of the street. These templates have two rollers at each end; the faces of the rollers are slightly wider than the curb thickness. They are swivelled so that one end of the template can be drawn in advance of the other, thus accommodating a varying width of carriage-way of as much as 6 ins. The sand cushion is formed with great nicety by this means; it is then dampened, the bricks are laid and afterward rolled with a 5-ton roller. The pavement is then carefully inspected and imperfect

Mr. Andrews. bricks are replaced. The sand cushion is hard and compact, and the contact with the bricks is perfect. The joints are then grouted with Portland cement and sand in equal parts. The writer has laid 13 miles of brick pavement in this manner and has observed no rumbling or distortion such as has been the cause of complaint in some cities. The bricks used have been red shale in all cases.

Mr. Owen's suggestion of a depression in the center of the street is worthy of serious consideration. The writer has often been impressed with the desirability of such a form of cross-section when for several months the gutters are filled with snow and ice, and in times of thaw all drainage is necessarily in the center of the street. Except in the case of alleys where there are no sidewalks the writer has, however, never used a cross-section throwing the water toward the center of the carriage-way.

A low retaining wall $1\frac{1}{2}$ to $2\frac{1}{2}$ ft. high has been used with advantage in place of a curbstone on the high side of a carriage-way. In Troy, N. Y., there exists such a wall nearly 6 ft. in height.

In the improvement of the streets of the older and of the hilly cities the municipal engineer is often in despair at the difficulties arising from the original and grossly improper formation of the street. The author is to be congratulated on the comparatively easy nature of the problem and on the readiness of the inhabitants, as commendable as unusual, to co-operate with him in the carrying out of his plans.

Mr. Woods. ROBERT P. WOODS, Jun. Am. Soc. C. E.—It appears probable that methods of street construction may vary in different parts of the country and still be suited to their respective localities and attending circumstances.

The method of shaping unbalanced cross-sections recommended by Mr. Tillson was used by the author on Miami Street (Sections 1 and 2), and would have been used throughout the improvement, but that its continued application would have produced results apparently more unsatisfactory than the double curb.

The five sections as rearranged by Mr. Tillson are not as practical as might appear from a casual inspection of them singly. In his Section No. 2 the east gutter on Miami Street is raised 10 ins., leaving but 2 ins. fall in a block of 280 ft. This, obviously, would not answer.

By raising the west gutter on Wabash Street 1 ft., as shown in his Section No. 5, the grade of the gutter at the alley north of Canal Street would be 1 in. above the sidewalk.

In his Section No. 7 the slope of the south walk on Market Street is lessened to 0.20 ft., which is sufficient for that walk; but the longitudinal grade of the west walk on Wabash Street is not as great as the Market Street walk, and he thereby reduces the cross-slope of the Wabash walk to 0.08 ft. or $\frac{1}{13}$ in. per foot, which is not enough.

Mr. Woods.

The predominating slopes used by Mr. Tillson in the roadway revisions appear to be far too slight for the existing conditions in Wabash, and from this standpoint the author believes they are not acceptable.

The author's Sections Nos. 7 and 8 show the slope of the roadway to be from one gutter to the other, but in connection therewith it should be noted that either of these sections is on a warped or boned-in section, and was exhibited merely to show a partial detail of the manner in which the Market Street grade was merged into the hill-side grade of Wabash Street. At no place in the improvement does water drain transversely of the roadway from one gutter to the opposite; the longitudinal grade of the north gutters on Market Street, at the warped sections cited, each being 3%, the water is carried to them.

It has been suggested that the extra step should have been placed at the building line, or at the area line. To illustrate the local feeling concerning such location, it may be stated that the original grade on the north walk of Market Street from Wabash Street westward 324 ft. was flush with the lower curb; but before this particular walk was constructed the nine abutting property owners signed a petition asking the Common Council to raise the grade by means of the double curb, and at the expense of the abutters. This petition, which had been presented after most of the double-curb work had been constructed, would lead one to presume, that from a local standpoint the curb was not considered "expensive, unsightly and dangerous," as indicated by Mr. Tillson.

The total extra cost of the improvement to the abutting property owners adjacent to the double curb was but 10 cents per foot more than to those along the single curb.

Mr. Evans believes the double curb would be a repository for filth, yet during the nine months that have elapsed since the construction it has not been, nor should it become so, for should the merchants fail to sweep the curb when they sweep the walk, the drainage from the walks, upon the smooth cement surface of the curb, which has a cross-slope of $\frac{1}{4}$ in. per foot, would be ample to keep it in a sanitary condition.

Mr. Owen's preference for a single high step at the curb, when it is necessary, is well founded where there is to be much transfer of heavy merchandise; but in Wabash the freight is transferred from the wagons to the buildings in the 16-ft. alleys.

It was the peculiar local conditions surrounding the district improved that caused the author to arrange the double curb as used, and the results are decidedly satisfactory.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications

DRY DOCKS—STONE vs. WOOD.

An Informal Discussion.*

By EDWARD MOHUN, M. Am. Soc. C. E.

Mr. Mohun. EDWARD MOHUN, M. Am. Soc. C. E. (by letter).—The writer is thoroughly in accord with those who hold, that while a commercial company, or private individual, may be justified, from financial considerations, in the use of timber for these works, yet that a great and wealthy maritime nation would be ill advised if, through the parsimony of its rulers, other than the best and most enduring material were made use of in constructions on which in time of war the efficiency and possibly the very existence of its fleet might depend.

The dry dock at Esquimalt, B. C., was opened in July, 1887; H. M. S. *Cormorant* being the first to enter.

Its dimensions are: Length of floor, 450 ft., which can be increased to 481 ft., by berthing the caisson at the outer invert; breadth of entrance, 65 ft.; breadth at coping, 90 ft.; breadth of floor, 41 ft.; depth on sill at ordinary high water, 26½ ft., and at high spring tides from 29 to 30 ft.

The body of the dock is constructed of concrete faced with masonry.

From the bills of quantities there were used in its construction: 30 639 cu. yds. of concrete, 6 556 cu. yds. of brickwork, and 109 887 cu. ft. of masonry.

Its cost was about $900 000.

* The discussion of this subject, for which no formal paper was presented, is here continued from the *Proceedings* of March, 1899, in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publications, *Proceedings*, Vol. XXV, p. 71.)

Mr. Mohun.

It has been in almost constant use for nearly 12 years, and the writer is informed by the Dock Master that the repairs to the body of the dock, during that period, have been effected at the cost of five barrels of cement and a week's wages to a mason.

During the last years the cost of replacing defective work on the caisson chamber and pump wells has been about $4 000. Owing to the records having been destroyed by fire, it is impossible to give the correct figures. Much of this expenditure appears to have been caused by the use of brickwork in the pump wells and entrance works. The writer regrets his inability at the present moment to furnish the actual cost of these repairs, but it appears reasonable to assume that, had the same material been used as in the body of the dock, their cost would have been reduced to a minimum.

The pumps have been furnished with two new sets of rubber valves, and a new pinion and spur wheel have been added to the machinery.

Irrespective of the tendency, of late years, to increase the size of warships, it would seem, that to fulfill all the conditions under which its services might be needed, a dry dock should be long enough to take the longest vessel in the fleet, wide enough at the entrance to take that of the greatest beam, and with depth on the sill to take that of the greatest draught.

The following are the dimensions of the largest ships in the British Navy:

Battle-ships, *Majestic* class: Length, 390 ft.; beam, 75 ft.; draught, $27\frac{1}{2}$ ft. There are nine of this class.

The *Powerful* and the *Terrible* are the two largest cruisers of the British Navy afloat, and have a length of 500 ft., a beam of 71 ft., and a draught of 27 ft.

It appears to the writer that the least dimensions of dry docks for the use of the British Navy at outlying stations would be: Length of floor, 500 ft.; width at entrance, 80 ft., and depth on sill, 30 ft. Probably few engineers would recommend the construction of timber docks of such dimensions to receive battle-ships of 15 000 tons displacement.

A few weeks ago the writer was discussing with the captain of the flagship on this station the size of docks required by the later additions to the British Navy, and that gentleman pointed out that under certain possible conditions a very great depth on the sill might be needed. It must be understood that this was simply in conversation and possibly might not be the result of mature consideration on his part.

For instance, suppose a vessel with her bows stove in and her forward compartments full of water, the result perhaps of a naval action or collision; it is obvious that she would be down by the head,

Mr. Mohun. possibly 10 ft. more than her normal draught, and to enter the dock would need a great depth on the sill. The officer above referred to suggested that a depth of 40 ft. on the sill might be required in such an emergency.

To the writer this is an entirely new view, on which he does not feel competent to express an opinion.

A case somewhat in point, however, occurred here a few years ago. The second-class cruiser, *Amphion*, 4 300 tons, with a usual draught of 22½ ft., was taken into dock after ripping up her bottom on a rock. She reached the dock at high water, and was just able to enter, grazing the sill.

The Admiralty grant toward the construction of the Esquimalt dock was $250 000, and this single use of the dock probably recouped the amount and possibly saved a vessel to the British Navy.

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COMPARATIVE TESTS OF BITUMINOUS STEAM COALS.

Discussion.*

By WILLIAM KENT, Esq.

WILLIAM KENT, Esq.—This paper is a valuable contribution to the knowledge of the heating value of some of the most important American coals, and Mr. Hill is to be congratulated on having presented one of the most important papers on this subject which has yet appeared. Mr. Kent.

The author has had an opportunity which few engineers have ever obtained of making a systematic series of tests of coals mined over a wide extent of territory, with conditions under his control, and with facilities for making both chemical and calorimetric tests.

All the results which the opportunity offered have not been obtained by the author, nor has he drawn all the conclusions which might have been drawn, and the speaker will have to find fault with some details of his methods and with some of his figures.

The map, Fig. 2, showing the location of the coal fields, is especially to be commended. The chief factor which determines the heating value of any of the coals mined east of the Mississippi is the geographical location of the mine. No satisfactory study of American coals can be made without a map showing the location of the mines.

*This discussion (of the paper by John W. Hill, M. Am. Soc. C. E., printed in *Proceedings* for April, 1899), is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. XXV, p. 71.)

Communications on this subject received prior to June 23d, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

Mr. Kent. In regard to the methods of testing, the author was fortunate in having an internally fired boiler which could be driven at a uniform rate and thus keep the loss by radiation practically constant. He was unfortunate, however, in having the rate of evaporation between 5 and 6 lbs. per square foot of heating surface per hour, so high as to be far beyond the point of maximum economical performance. He was also unfortunate in not having a furnace and combustion chamber surrounded by fire brick, which would have been of great assistance in burning completely those coals which had over over 30% of volatile matter.

The author says:

"The results given in the tables are in no case supposed to indicate the best possibilities with these coals. * * * The author regards the results obtained by this method of handling the coal as more satisfactory to the buyers and users of coal than those which might have been obtained by expert firemen."

The results are certainly below the best possibilities of the coals. They could not be anything else, with the boiler driven beyond its economical rating, with a furnace not adapted for burning the volatile gases, and with only the ordinary firemen. This may be satisfactory to the buyer and user of the coal, but it should not be satisfactory to the seller of the coal nor to an engineer. The object of the tests appears to have been to determine the relative value of the coals. If each coal had been tested in such a manner as to develop its best possible results, with a clean boiler, with expert firemen and with a boiler not over driven, the results then could be taken as expressing the true relative value of the different coals. But when conditions are unfavorable, as they were in these tests, there is no way of determining that they are not more unfavorable for one coal than for another. Unfavorable conditions are variable conditions, and they should be avoided in testing coals, boilers and engines, just as much as in racing horses.

An example of the effect of what was probably variable firing is seen in the figures for Pittsburg coal, for the item "pounds of steam from and at 212° per pound of combustible" in Tables Nos. 5 and 6.

Name.	Lump.	Barge Run.	Mine Run.	Nut and Slack.
Brown	9.54	9.64	9.25	9.24
Catsburg	9.87	9.46	9.36	9.14
Cincinnati	....	....	9.24	9.34
Woods	9.65	9.43	9.90	9.43

There can be no doubt that these coals were practically of the same quality, and it might be expected that these 14 tests would give some knowledge of the relative value of different sizes. But the highest figures are given by the Barge Run for the Brown coal, by the Lump

for the Catsburg, by the Nut and Slack for the Cincinnati, and by the Mine Run for the Woods. The Brown coal is about 2% better than the Woods, if judged from the Barge Run tests; but 7% poorer than the Woods, if judged by the Mine Run tests. Mr. Kent.

The author has something to say concerning a curve which the speaker plotted from the results of Mahler's experiments in France in 1892. *

The speaker will not undertake at present a defence of this curve, but will only say that the results presented in the paper have not caused him to modify in any degree the latest statements he has made in relation to it, giving the limitations of its usefulness.

The results given in the paper will now be taken up and a few conclusions drawn therefrom.

The data give three different ways of estimating the probable heating value of the several coals: (1) The determinations by the Carpenter calorimeter, (2) the approximations by the use of the speaker's curve, and (3) the results of the boiler tests. Each of these three methods is subject to error.

The Carpenter calorimeter is by no means recognized as being as accurate an instrument as the Mahler calorimeter. It is greatly to be regretted that Mr. Hill did not have his results checked by Professor Lord, at Columbus, with his Mahler calorimeter. The standardizing and the manipulation of the Carpenter calorimeter is a matter of great delicacy, and any work done by it, to have scientific value, should be done by an expert.

The use of the speaker's curve depends on accuracy in the proximate analysis of the coal, which is not easy to obtain, as may be found by sending duplicate samples of coal to three or more chemists for analysis. The speaker does not claim for his curve a greater approximation to accuracy than 3%, even when the analysis is accurate; and it is liable to a somewhat greater error than this in coals that are high in volatile matter, such as the Pittsburg and Thacker coals.

The best boiler tests are subject to an error of about 3%, and, as already shown, these particular boiler tests show some evidence of having even larger errors, or, more properly, larger variations from the results which were possible under the given conditions.

There is a strong probability that the efficiency of the boiler, barring unavoidable variations due to irregularity of firing and the ordinary errors of a test, is approximately uniform for all the coals of a given class, and if we assume a certain efficiency of boiler for these tests, we will arrive thereby at an approximation to the heating value of the coal. There can be no such error in this method as there is apparently in the calorimetric results obtained by Mr. Hill. For

* "Mineral Industry," Vol. I, p. 97, and revised with the addition of the results of Lord and Haas, in 1898; *Transactions*, A. I. M. E., Vol. xxvii, p. 955; and *Transactions*, A. S. M. E., Vol. xx.

Mr. Kent. instance, in Table No. 23 the efficiency of the boiler when testing Loup Creek, Mine Run coal was only 57.51%, while it was 69.09% with the Belmont Splint, Mine Run, or over 20% higher. This is only partly explained by the poor result of the boiler test with the Loup Creek coal, and it must be largely due to an incorrect calorimetric result with the Belmont coal.

It may be assumed, for the purpose of this study, that the three different estimates of the heating value of these coals are of equal credibility, and that a somewhat near approximation to their true heating value can be obtained by averaging the three estimates. To obtain a convenient approximation to the heating value of the coal from the boiler tests, the speaker has multiplied the figures representing the evaporation per pound of combustible by 1 500 for the Pocahontas, New River and Loup Creek coals, and by 1 600 for all the other coals. This is equivalent to assuming a boiler efficiency of 64.38% for the three coals first named, and 60.36% for the other coals*, which is a fair assumption, since the three coals are of a distinct class, semi-bituminous, and always, in internally fired boilers, give higher efficiencies than the bituminous coals.

From the several results the speaker has selected the best obtained from each coal, whether Lump, Barge Run, Mine Run, or Nut and Slack, thus giving each coal the advantage of the best showing it made. This seems to be only fair to each coal, and it does not seem fair to depreciate the value of Loup Creek, which gave 10.63 lbs. evaporation in Lump coal by crediting it with the extremely low figure of 9.13 for the Mine Run coal, which was no doubt due to improper firing.

Making the calculations in this way, the results given in Table No. 29 have been obtained.

It will be noticed that the average figures in column 4 show a much higher degree of uniformity than the figures in columns 1 and 3, and a somewhat higher degree of uniformity than the figures in column 2. The last three columns show the differences of columns 1, 2 and 3 from the average. Comparing these figures of differences the following results are found: Of the twenty-three coals the maximum difference between the average and the three individual results for each coal occurs 15 times in column 5, representing the variation of the Carpenter calorimeter results from the average; once in column 6, representing the variation of the results obtained by the Kent curve from the average; and 7 times in column 7, representing the variation between the boiler test results and the average. The minimum difference, or the nearest approximation to the average value, occurs 7 times in column 5, 11 times in column 6, and 5 times in column 7. The

*10 lbs. evaporation $\times$ 1 500 = 15 000 B. T. U.
$10 \times 965.7 \div 15\,000 = 64.38\,\%$.
10 lbs. evaporation $\times$ 1 600 = 16 000 B. T. U.
$10 \times 965.7 \div 16\,000 = 60.36\%$.

average difference is 418 in column 5, 239 in column 6, and 339 in column 7. Mr. Kent.

Thus, by three different methods of comparison the Kent curve shows a closer approximation than either the calorimeter or the boiler test to the average heating values obtained from the three different methods of estimation.

TABLE No. 29.—B. T. U. per Pound of Combustible.

	1	2	3	4	5	6	7
	By Calorimeter.	By Kent's curve.	By boiler test.*	Average.	Average greater or less than		
					1	2	3
Pittsburg, Brown, 2d Pool	14 381	15 670	15 427	15 159	778	—511	—268
" Catsburg, 3d Pool	14 317	15 260	15 784	15 120	803	—140	—664
" Cincinnati, 3d Pool	14 036	15 270	14 950	15 073†	1 037	—197	123
" Woods, 4th	14 213	15 350	15 847	15 137	924	—213	—710
New River	15 757	15 840	16 167	15 921	174	81	—246
Loup Creek	15 336	15 845	15 950	15 710	374	—135	—240
Pocahontas	15 456	15 840	15 905	15 734	278	—106	—171
Kanawha Steam	15 653	15 470	15 909	15 677	24	207	—232
" Gas	15 440	14 780	15 766	15 329	—111	549	—437
Acme	15 589	15 460	16 296	15 782	193	322	—514
Keystone	15 448	15 490	15 501	15 480	32	— 10	— 21
Winifrede	14 693	15 375	15 853	15 307	614	— 68	—546
Thacker	15 554	15 400	16 205	15 720	166	320	—485
Logan, Cons. Thacker	14 968	15 345	15 686	15 333	365	— 12	—353
Maritime, Thacker	15 008	15 380	15 487	15 292	284	— 88	—195
Campbell's Creek	14 789	14 690	15 455	14 978	189	288	—477
Monongah	15 022	15 300	15 331	15 218	196	— 82	—113
Eagle	14 625	15 500	15 798	15 308	683	—192	—490
Cedar Grove	15 422	15 120	15 746	15 429	7	309	—317
Mt. Carbon	14 761	15 380	15 322	15 154	393	—226	—168
Chesapeake	14 086	15 210	14 921	14 739	653	—471	—182
Coalburg	14 686	14 510	15 050	14 749	63	239	—301
Belmont Splint	13 262	15 275	15 076	14 538	1 276	—737	—538

*Calculated by multiplying the pounds of steam from and at 212° by 1 500 for the New River, Loup Creek and Pocahontas coals, and by 1 600 for all the other coals. This is equivalent to assuming a boiler efficiency of 64.5% for the three coals named and 60.4% for the other coals.

†Average of columns 2 and 3 only, the figures in column 1 being considered incredible.

The speaker appends the following remarks, as showing the conclusions he has reached from earlier studies in relation to the heating values of semi-bituminous and bituminous coals mined in different districts, which conclusions are confirmed by this study of Mr. Hill's paper.

Draw a line northeasterly from the Pocahontas field, shown at No. 6 on the map, Fig. 2, on the boundary line between Virginia and West Virginia, through Maryland a little west of Cumberland and through Somerset, Cambria and Clearfield counties in Pennsylvania. This line, 250 miles in length, will pass through the coal fields containing

Mr. Kent. the best steam coals mined in the United States. A remarkable feature of these coals is their uniformity in chemical composition, their combustible portion varying only within 2 or 3% from 80% fixed carbon and 20% volatile matter. The heating power is likewise remarkably uniform, averaging 15 750 B. T. U. per pound of combustible, with a probable maximum variation of not over 2% on each side of this figure. The ash and moisture of all the coals along this line are also very low, the ash varying no doubt with the care used in preparing the coal for market at the individual mines. These coals are properly classed as semi-bituminous.

Traveling northwestwardly, at right angles to and from any point on this line, it is found that the coal gradually increases in volatile matter, and decreases in heating value. In the coals mined on the line the volatile matter is only about 20% of the total weight of the combustible, but it is all nearly of the composition of marsh gas, $C H_4$, having but little oxygen; and every pound of it is worth nearly $1\frac{1}{2}$ lbs. of carbon. Further to the northwest, however, while the proportion of volatile matter increases, it becomes of poorer quality, being higher in oxygen. In some parts of Ohio the volatile matter is worth no more per pound than carbon, and in most Illinois coals it is worth less.

Referring to the map, Fig. 2, it is found that, with the exception of the Jackson, Ohio, and Jellico, Kentucky and Tennessee coals, of which the author had only a few samples, nearly all the coals tested by him were mined not very far distant from the northeast and southwest line described. The Pocahontas, New River and Loup Creek coals are on the line, and their heating value averages approximately 15 800 B. T. U. per pound of combustible. The Pittsburg, Monongah, Winefrede and Kanawha, and Thacker fields lie at some distance to the northwest of the line. The Pittsburg coals are the farthest away from the line, and their probable average heating value is 15 100 B. T. U. All the other coals, of the fields previously named, from Fayette, Kanawha and Mingo counties, West Virginia, from sixteen different mines, scattered along a northeast and southwest line over 150 miles in length, have a heating value the entire range of which, (omitting the Belmont Splint coal, the apparent average heating value of which is no doubt too low), is only between the figures 14 739 and 15 782 B. T. U. per pound of combustible. The average of these figures is 15 260 B. T. U., and the range on either side of this average is only 3.4 per cent. This result is a confirmation of the conclusion reached by Professors Lord and Haas in their paper in the *Transactions* of the American Institute of Mining Engineers, to the effect that the coals mined in a given seam, over a considerable area of territory, have a substantially uniform heating value per pound of the combustible portion.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will hereafter be reproduced in the Volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WATERMAN STONE, Assoc. Am. Soc. C. E.*

DIED MARCH 30TH, 1896.

Waterman Stone was the eldest son of Lemuel M. E. Stone, one of the engineers engaged in early railroad work in New England, and was born in Cumberland, R. I., March 10th, 1847. His engineering training was under his father's direction. In 1871 he succeeded his father as Superintendent and Engineer of the Providence, Warren and Bristol Railroad, and during the eighteen years he held this position he designed and supervised all the improvements made on the road. The most notable of these was the construction of a curve of 211 ft. radius in the main line, just outside the City of Providence, R. I., to enable trains to enter and leave the city head on. A bluff 130 ft. high rose from the East Providence shore of the Blackstone River at the point where the railroad crossed, and since the construction of the road, trains had been run on a Y-track above the bridge, and were pushed in and out of the city. Mr. Stone put in a 27° 20′-curve between the river's edge and the bluff. A modified Fairlie type of locomotive was built to haul trains around this curve and standard eight-wheel American type locomotives were also run over it. The forward pair of driving-wheels of such engines was equipped with blank tires with a width of tread of 7 ins., and third and fourth rails were laid on the curve for the blank tires.

On the absorption of the Providence, Warren and Bristol Railroad by the Old Colony system, Mr. Stone removed to Kansas City, Mo., where he became Superintendent of the Inter-State Rapid Transit Railroad. This was a cable railway, but during the four years of his service electric traction was installed. Returning to the East, Mr. Stone began practice as a consulting engineer, with especial reference to electric railroads. He had built several roads and was engaged upon others at the time of his death.

Mr. Stone was a man of sterling character, and his kindly disposition and love of fair play made him as popular with his employees as with his more general acquaintance. Of mathematical tendency, he was an earnest student in fields not bounded by his professional practice. He was elected an Associate of the American Society of Civil Engineers, December 1st, 1886. While in the West he was Secretary and Treasurer of the Engineers' Club of Kansas City, and he was the first Secretary of the American Society of Railroad Superintendents.

* Memoir prepared by the Secretary from information furnished through E. P. Dawley, M. Am. Soc. C. E., and from papers on file.

WILLIAM GARNETT WILLIAMSON, M. Am. Soc. C. E.*

DIED AUGUST 2D, 1898.

William Garnett Williamson, eldest son of General Thomas H. and Louisa Garnett Williamson, was born in Norfolk, Va., December 13th, 1840. He attended Fuller's Classical School, of Lexington, Va., and then entered Washington College (now Washington and Lee University), Lexington, Va., from which he was graduated in 1859, with the degree of B. A. He was afterward privately instructed in Civil Engineering by his father, who, for nearly fifty years, was Professor of Engineering at the Virginia Military Institute. His first service as an engineer was with General Robert E. Rodes upon railroads in Alabama. At the breaking out of the Civil War he volunteered in the Confederate Army as a private in the college company, but was soon transferred to the Rockbridge Artillery, and served with that battery under Captain, afterward General, W. N. Pendleton until he was appointed a Lieutenant of Engineers and assigned to the command of General Stonewall Jackson. Captain Pendleton was the Rector of Grace Church, Lexington, and some of Mr. Williamson's earlier military service was thus with his friend, his pastor and the instructor of his boyhood. As First Lieutenant of Engineers, Mr. Williamson was assigned to the First Regiment of Engineer Troops, C. S. A. (commanded by Colonel T. M. R. Talcott), of the Army of Northern Virginia. He was subsequently promoted to the Captaincy of Company F of that regiment, and served with it until the surrender at Appomattox Court House, where he was paroled.

After the war Mr. Williamson went to Mexico and there engaged in the construction of the railroad between Vera Cruz and the City of Mexico, of which Colonel Andrew Talcott, father of his regimental commander, was the Chief Engineer. Returning to Virginia in 1867 he was for six years engaged in railroad work in Virginia, North Carolina and Kentucky. From 1871 to 1873 he was in charge of the construction of shops, depots and yards, for the Chesapeake and Ohio Railroad, at Huntington, W. Va., and was Division Engineer of the road. From 1874 to 1876 he was Assistant Engineer on the Tennessee River surveys. Returning to railroad work, a year was passed in the service of the Buchanan and Clifton Forge Railroad in Virginia, as Division Engineer, and in 1878 he was again in the Government service as Assistant Engineer on survey work in Currituck Sound, N. C., the Nattoway, Chowan and Tar Rivers. In 1879–1880, he was with the Richmond and Danville (now the Southern) Railroad as Engineer of Bridges, under Colonel Henry T. Douglas, M. Am. Soc. C. E., with whom he was associated for several years. From 1880 to 1882 he was Assistant Engineer with the late General Quincy A. Gillmore, M. Am.

* Memoir prepared by the Secretary from information furnished by W. E. Cutshaw, William Cain and Geo. G. Earl, Members, Am. Soc. C. E., and from papers on file.

Soc. C. E., in the United States Survey of the Altamaha and Canoochee Rivers in Georgia, and later was Assistant Engineer on the survey of the Cahaba Coal Field, in Alabama.

He next undertook the construction of a sewerage system for Montgomery, Ala., from the plans of Rudolph Hering, M. Am. Soc. C. E. After the completion of these sewers, in 1883, he was elected City Engineer of Montgomery, and inaugurated a plan of improvements for that city. The work of a city engineer did not suit his sensitive nature, and in 1888 he resigned this position and again entered the government service as United States Civil Assistant Engineer on the Alabama River under Captain Hoxie, where he remained about one year. In 1888 he opened an office in Montgomery, Ala., for general engineering work, in which venture George G. Earl, M. Am. Soc. C. E., was associated with him. The firm continued for only about two years, during which time sewer design and construction and water-works extension for Americus, Ga., and sewerage plans for Columbus, Ga., were undertaken. Mr. Williamson had other work in hand at the same time, including sewerage plans for Sheffield, Ala., and the improvement of the grounds of the Alabama State Capitol. Government work and surveys, where he could be surrounded by friends, were always attractive to Mr. Williamson, and hence, in 1891, he went with the United States Engineers on river and harbor work again. He was engaged under Major F. A. Mahan as United States Civil Assistant Engineer on the Coosa River, where he remained until the war with Spain began, when he was transferred to Pensacola, Fla., and had charge of work upon fortifications and harbor defences. While upon this work the climate and overwork brought on an attack of dysentery which resulted in his death.

Mr. Williamson was descended from English ancestors through a line of distinguished Virginians. He was an engineer of more than usual ability, both in theory and in practice, a better mathematician than most engineers of ante-bellum college training, precise in everything and an excellent draftsman. He was devoted to his profession and untiring in his duties, a man of varied experience, and, in the words of one who was associated with him, "so honorable in every relation of life, particularly professionally, that there would be no need of a code of ethics in the profession, were all its members but one-half so far above temptation as was Captain Williamson." Another engineer writes of him as "one of the most open-hearted and lovable of men, honest and true to a fault, and, withal, a man of uncompromising principle." He married Miss Bettie Cralle, of Huntington, W. Va., October 10th, 1873, and she and seven children survive him.

Mr. Williamson was elected a Member of the American Society of Civil Engineers April 4th, 1883.

THOMAS SPARKS BISHOP, M. Am. Soc. C. E.*

DIED NOVEMBER 13TH, 1898.

Thomas Sparks Bishop was born March 12th, 1846, near Columbus, Burlington County, N. J. His father was a farmer, and his early educational facilities were such as were usually afforded by the ordinary country public and private schools. Before entering college he was a pupil at Westtown Boarding School near Westchester, Pa.

He entered the Philadelphia Polytechnical College September, 1864, and was graduated as Civil Engineer in June, 1866. The Master's degree was conferred upon him in 1869. Early in the succeeding August he was appointed Rodman on the engineer corps of the Camden and Amboy Railroad, and was engaged in the location and construction of the Pemberton and Hightstown, and Camden and Burlington County railroads until their completion, about a year and a half later.

In March, 1868, he was appointed Assistant Engineer of the Freehold and Squankum Marl Company's Railroad, after the location of which, in June of that year, he was recalled to the main line of the Camden and Amboy Railroad in the capacity of Assistant Engineer. In addition to his other duties there, he had charge until January, 1872, of the location and construction of various branch railroads. During this time, however, he was detailed as Resident Engineer to take charge of the construction of the inclined plane machinery, etc., of the Bingham Mining and Lumber Company at Renovo on the Philadelphia and Erie Railroad, where he was engaged from September to December, 1868.

In January, 1872, after the lease of the works of the United Companies by the Pennsylvania Railroad Company, he was appointed Assistant Engineer of Construction on the United Railroads of New Jersey Division, and in December of that year was assigned as engineer in charge of the improvements at Harsimus Cove and Bergen Hill, at Jersey City. Upon their completion in January, 1875, he was transferred to Trenton, N. J., where he made re-surveys of the various railroads of the United Railroads of New Jersey Division for the real estate and maintenance of way departments, until May, 1880, when he resigned to accept an appointment as engineer with the Russell & Erwin Manufacturing Company at New Britain, Conn. He served in that capacity for eighteen years, resigning his position June 1st, 1898, to devote his attention to private practice as a civil engineer and surveyor, with headquarters at New Britain, Conn.

Mr. Bishop was married in November, 1873, and was elected a Member of this Society September 2d, 1885.

* Memoir prepared by T. Sparks Bishop, Esq., from autobiographical data left by his father.

FREDERICK HENRY SMITH, M. Am. Soc. C. E.*

Died December 24th, 1898.

Frederick Henry Smith was born in Pittsburg, Pa., on November 10th, 1839, and there his early schooling was effected. At the age of thirteen he went with his uncle, Col. James Shaler, to Lake Superior, where, as a boy, in immediate contact with some of the great mining industries, he acquired his taste for geology and mineralogy. When but sixteen years of age he entered the engineering service of the Louisville and Nashville Railroad, under the late George MacLeod, C. E., and continued in this field until the breaking out of the Civil War, when he became a military engineer in the service of the Confederate States.

Here, besides serving on strictly military duty, he was detailed to the Niter and Mining Bureau, where he devoted his energy to the production of saltpeter and other material required for munitions of war. The caves of Alabama, Georgia and Tennessee were ransacked, under his guidance, for niter to supply the powder-mills. He was also engaged during the war in rehabilitating some of the old and disused iron-smelting furnaces, and in other metallurgical work.

At the close of the war, Captain Smith turned his varied experience to the practice of civil engineering, and, establishing himself at Baltimore, Md., became a member of The Baltimore Bridge Company. This connection was terminated in 1873, Captain Smith continuing his practice at Baltimore, and devoting much of his time to the design and erection of bridges. He became Engineer of Bridges to the City of Baltimore, and Consulting Engineer for the railroads of the Seaboard Air Line. In both these capacities he executed some excellent examples, among others the handsome stone arched North Avenue Viaduct, in the City of Baltimore, which is likely long to remain as a monument to his professional ability.

His interest in geology and mineralogy led him to employ such brief intervals of leisure as came to him in the preparation of a popular hand-book, entitled "Pocket Geologist and Book of Minerals," which he published in 1877. This was revised and republished in 1882 as "Rocks, Minerals and Stocks." In 1890, he published the "Pocket Geologist and Mineralogist," and in 1895, "Smith's Hand-Book of Valuable Minerals: What They Are; What They Look Like; Where To Look for Them," a duodecimo of 238 pages. These small volumes are full of useful and practical information for the prospector and miner, and contain in pocket form much valuable matter in relation to rocks and minerals.

* Memoir prepared by Mendes Cohen and C. H. Latrobe, Members, Am. Soc. C. E.

During the last year of his life, he was the engineer member of the State Board of Health of Maryland, where his general knowledge and experience made him an efficient member, whose opinions were highly valued by his colleagues.

Mr. Smith's health broke down completely several years before his death, but notwithstanding constant suffering, his fortitude, energy and patient good humor, sustained him to the last. He became a Member of the American Society of Civil Engineers on February 21st, 1872, and although he never took a very active part in the deliberations of the Society, yet he was always esteemed a valued member. He married on December 27th, 1870, Miss Elizabeth Gairdner, of Augusta, Ga., who, with two sons and one daughter, survives him. He passed away quietly and peacefully, much regretted by those who who knew him best.

SAMUEL KILLEBREW, M. Am. Soc. C. E.*

Died January 9th, 1899.

Samuel Killebrew, son of George W. and Mary A. Killebrew, was born near Oak Grove, Christian County, Ky., May 14th, 1840. From early childhood he manifested a taste for mathematics, which his parents encouraged and cultivated, placing him under the care of the best teachers in the vicinity of their home and giving him a thorough preparation for college. He entered college at Russellville, Ky., in January, 1856, and remained two years, leaving, to assume his first work in the line of his chosen profession.

He began his career in engineering as rodman under Col. George B. Fleece, Chief Engineer of the old Memphis, Clarksville and Louisville Railroad in 1859. Mr. John B. Tapscott, who was in charge of the division on which he was employed, says of him:

"He was a country boy of rough exterior and looked as if he could manage a plow team or split a rail better than he could manipulate a transit or level. In less than two weeks I found my country boy was far from being a bumpkin, but was a close, observing, accurate and valuable assistant."

He continued in the work until the Civil War began, when he left the railroad to serve the Confederate Government. In 1870 to 1878 he was employed in various capacities in the location and construction of the Brownsville and Ohio Railroad in Tennessee, his continued advancement from rodman on preliminary survey to responsible charge of construction bearing evidence of his energy and ability.

In 1881 he was appointed Assistant Engineer of the Galveston, Harrisburg and San Antonio Railway, then being built from San Antonio, Texas, westward to a connection with the Southern Pacific, and was assigned to duty as instrument man in a locating party. It was on this work that the writer first met him, and here began a friendship which only terminated with his death. As is often the case when a new man joins a party that has been in the field for some time, Mr. Killebrew was not welcomed with much cordiality by some of the members of the party. On one occasion, the engineer in charge being on reconnoissance and Mr. Killebrew being in charge, there was an open revolt against his orders. The manner in which he handled this trouble showed a firmness that would not permit trifling, but withal a kindly spirit of forgiveness that won for him the good will and respect of all the party. Not long after this "the Major" was the most popular man in the party and he was never troubled again.

At this time it was found very difficult to get a location over a certain summit, with the adopted gradient, and while the party was

* Memoir prepared by E. B. Cushing, M. Am. Soc. C. E.

on this work, Major Killebrew was detached from the regular corps to assist in reconnoissance. With a corps of only two men, and with only a surveyor's compass and chain he worked out a line that was located and built upon. This was the opportunity that demonstrated his ability, and Major Converse, the Chief Engineer, gave him charge of a party on another location. During the year that followed he located a line to Fredericksburg, Texas, 70 miles, and from La Grange to Bastrop, Texas, 35 miles. Though the company for which these lines were located did not build them, yet other corporations have since constructed the roads. Notwithstanding they had the best engineering talent, their constructed line does not vary appreciably from Mr. Killebrew's stakes.

In 1882, when Mr. C. P. Huntington and his associates decided to build into Mexico, Major Converse was asked to recommend a man to locate the line. He sent Mr. Killebrew to New York, and Mr. Huntington, with his good judgment of human nature, at once employed him. This may be called Mr. Killebrew's life work and his monument. Beginning at Piedras Negras, Mexico, in the middle of a hot summer to locate a line through a foreign country, the inhabitants of which were not generally friendly to "Gringo" intrusion, the language, customs and laws being entirely new, and the water, food and forage supplies being meager, he was beset from the first with difficulties that were extremely trying. With no precedents to guide him and with imperfect and unsatisfactory maps to indicate his course, he started his work with the same confidence and cheerfulness he had displayed in former undertakings. Mounting his mustang pony, his little compass and hand level in his saddle bags, he bade us farewell and started for the mountains. When, in ten days, he returned the general line of the Mexican International Railroad, over the first mountain pass, as it is to-day operated, was determined.

This line, as constructed, has several mountain summits to overcome and stream crossings to make, the detail location of which involved some prompt decisions on general points, as well as nice adjustments as to details. In addition to this, the location was made through a country where adequate food and water supply was wanting. There were frequent depredations by roving bands of Indians or by natives disguised as Indians, who were worse than the originals. The question of properly maintaining a party was often as trying as the work itself. During all this time Mr. Killebrew never once lost his control of the situation, but anticipated and provided for probable emergencies, spending much of his own time long distances ahead, looking out the way, and between the preliminary corps and the locating party, which were generally 25 to 40 miles apart. His presence around the camp fire encouraged the men, and his wonderful grasp and memory of topography rendered easy the work of his

assistant engineers. He never permitted a party to leave a district between two controlling points until both the assistant engineer in charge and himself were satisfied that the line in detail was where it should be and no considerable improvement was possible.

On one occasion, the owners of the property and their consulting engineer were anxious to build the road as close as possible to a certain town. Mr. Killebrew abandoned all work ahead and ordered both his corps to the work of modifying the line, assigning one party to each side of the river and directing both in person. When the final determination was made, it was after every available route had been tried. The line, however, was not satisfactory, and as the construction advanced, other engineers were detailed to get one nearer in. Major Killebrew heard of this, but instead of being hurt, he said: "I hope they will get what is wanted, but I am afraid they will not." The road was built on the Killebrew line.

He remained with this company until all its projects had been explored and much of the line built. In 1889 he was employed by the Mexican Pacific Railway in a very difficult location on the Atlantic side of the mountains, in Southern Mexico. This location led to the head of navigation on the Grijalva River, at San Juan Bautista. A portion extended through a mountain range entirely unknown. and as the coast was approached, the route lay through swamps and a dense tropical jungle, in which life was made a burden by the swarms of stinging insects and the dangers of bites from venemous reptiles. His principal assistant, Arthur C. Wheatley, Assoc. M. Am. Soc. C. E., writing of this work says: "He was always cheerful and encouraging in spite of the dangers, difficulties and sickness which surrounded us." On the completion of this work he returned to his home in Brownsville, Tenn.

When the Mexican International decided on further extensions, Mr. Killebrew was again sent for and placed in charge of the surveys. He made some extensive reconnoissances and surveys, including lines from Durango to Guadalajara, Durango to Guanacevi, Durango to Mazatlan, and many others; his work for this company embracing, probably, between 2 500 and 3 000 kilometers of final location, much of which was built. He was engaged in surveying a line from Durango to the Pacific Ocean when the final sickness came on him, and he died in Durango, Mexico, January 9th, 1899.

A chronological statement of his work does not constitute a proper memoir of Samuel Killebrew, for he was an extraordinary man in many respects. He was very simple in his tastes and very approachable. He was genial in manner and would go out of his way to show attention to the people living along the line of his surveys. His judgment of topography was a natural gift, but one that he always cultivated. He could ride through a new country and estimate with a wonderful

degree of accuracy the differences in level and distance between the controlling points. In later years, his health was broken, but his energy prior to this was ceaseless. He never let anything divert him from his work, and he instilled in those around him such interest in the work in hand that it was a topic of general conversation around the camp fire at night. In this way, the work was made as pleasant as possible to those who had to bear the burdens. He was painstaking with his men, never losing interest in showing them how to solve the problems that came up daily on such work, and many an engineer will, with the writer, always feel grateful for his kindly interest and assistance in those days of early trials and hardships. He was liberal with his men, and even in early days, when working on a small salary, he would share his earnings with those in need as cheerfully as when later on he could better afford to do so.

He was loyal and faithful to the interests of his employers, and had the work been his own, he could not have given it more attention. As a man, he never outgrew his boyhood's reverence and affection for his parents, and often on trips he would speak with the greatest affection of his mother and tell anecdotes of his early training. His wife accompanied him on the survey whenever it was practicable for her to do so, and was with him when he died.

His work is a monument to his professional skill, and he has left in the hearts of those who were with him tender memories of his many acts of kindness which they will never forget.

Mr. Killebrew was elected a Member of the American Society of Civil Engineers, September 2d, 1885.

HISTORICAL SKETCH

OF THE

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At the Annual Meeting, January 19th, 1898, the following facts in regard to the subscription to this book were brought out:

Two thousand copies were printed; 300 were bound in full morocco, of which 216 have been sold at $10 per copy, the resulting net profit being $943.06. Seventeen hundred copies, which have been paid for, are still on hand, and the Board of Direction was requested to consider the propriety of offering to the membership these copies bound in a less expensive style and at a reduced price, the net proceeds to be applied to the building fund.

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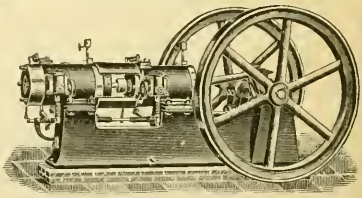

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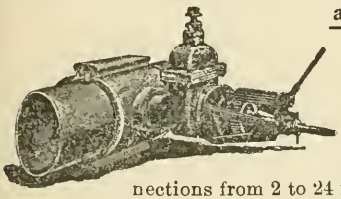

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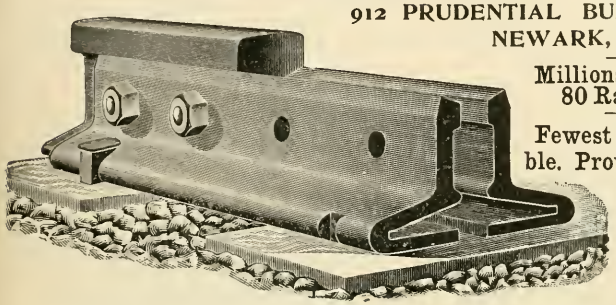

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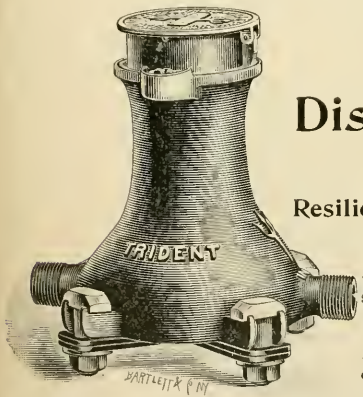

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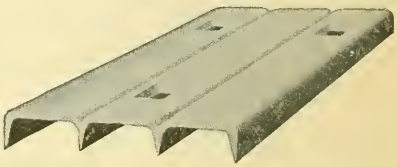

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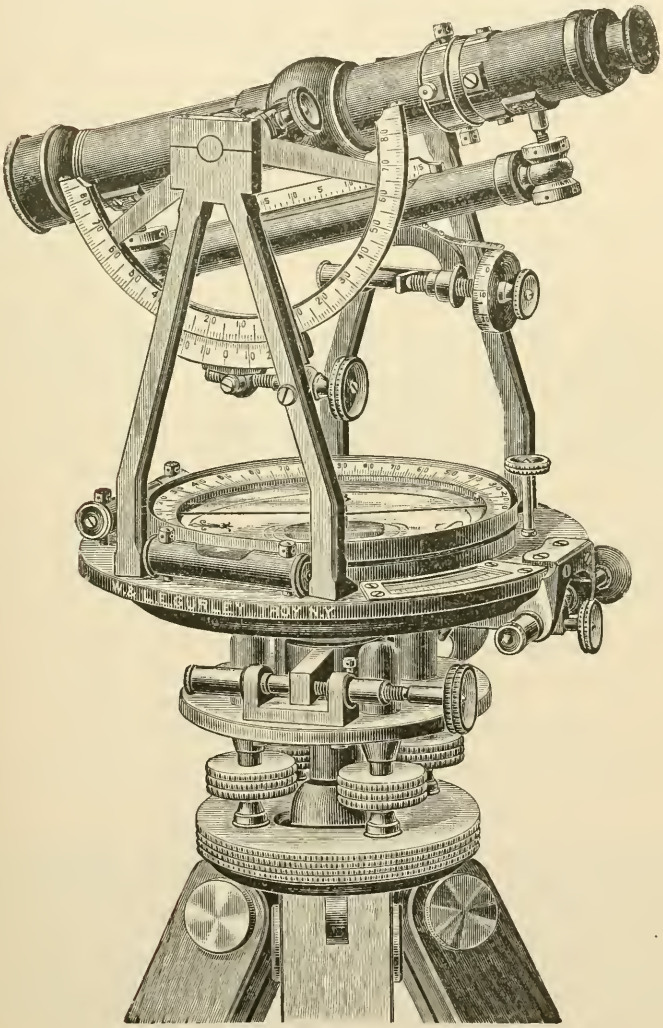

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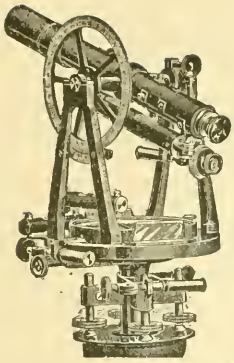

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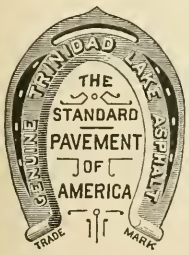

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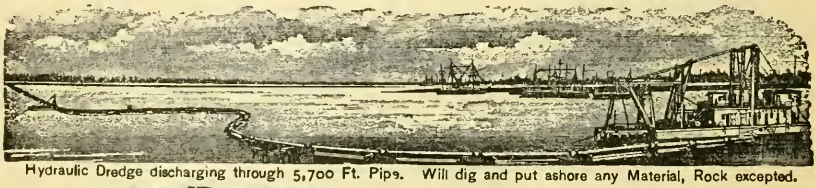

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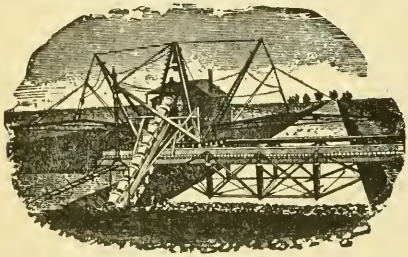

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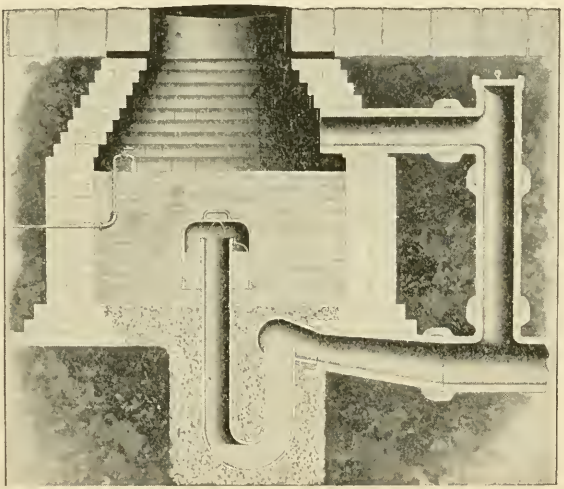

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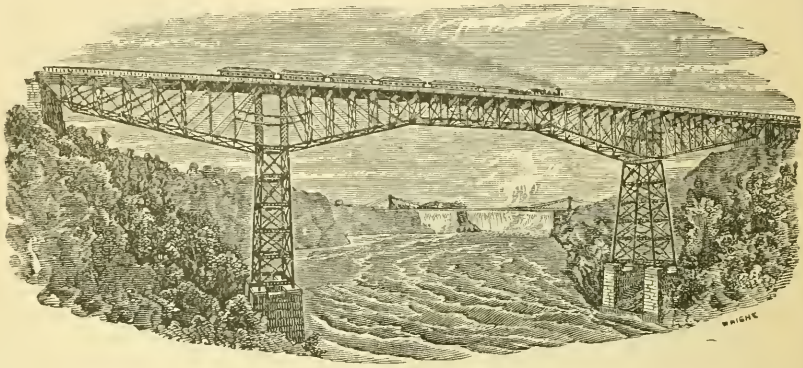

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OF

CIVIL ENGINEERS

August, 1899

PROCEEDINGS = VOL. XXV—No. 6.

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Vol. XXV. AUGUST, 1899. No. 6.

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publications.

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The prices of publications are as follows: *Proceedings*, $6 per annum; *Transactions*, $10 per annum. Postage will be added when they are sent to foreign countries.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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MINUTES OF MEETINGS.

OF THE SOCIETY.

June 7th, 1899.—The meeting was called to order at 20.35 o'clock, Vice-President Edward P. North in the chair; Charles Warren Hunt, Secretary, and present, also, 65 members and 11 visitors.

The minutes of the meetings of May 3d and 17th, 1899, were approved as printed in *Proceedings* for May, 1899.

A paper by George W. Rafter, M. Am. Soc. C. E., entitled "On the Theory of Concrete" was presented by the Secretary, who also read a communication on the subject from Past-President B. M. Harrod. The subject was discussed by Messrs. Herman Conrow, Allen Hazen, C. E. Fowler, G. W. Tillson, H. F. Dunham, E. Sherman Gould and P. W. Henry.

Ballots were canvassed and the following candidates declared elected:

As Members.

Linus Weed Brown, New Orleans, La.
Ralph Henry Brown, Boston, Mass.
William Warren Cummings, Everett, Mass.
Lewis Singer Gillette, Minneapolis, Minn.
Alvah Seymour Going, Victoria, B. C.
Eugene Griffin, New York City.
Albert James Himes, Detroit, Mich.
William Evelyn Hutchings, Louisville, Ky.
Henry Aiken Kennedy, Canton, Ohio.
Edmund Dana Trowbridge Myers, Jr., Richmond, Va.
Harold Parker, Clinton, Mass.

As Associate Members.

George Marshall Ames, Grand Rapids, Mich.
Robert Henry Persse Ford, St. Albans, Vt.
Wallace Greenalch, Schuylerville, N. Y.
Frederick Stuart Greene, New York City.
William MacLean Homan, Kroonstad, South Africa.
Francis Lansing Pruyn, Brooklyn, N. Y.
Spencer James Steward, New York City.
George Aymar Taber, New York City.
Horatio Hartley Trundle, Washington, D. C.
Henry Eleazer Vanderlip, Chicago, Ill.
Joseph Christian Wagner, Philadelphia, Pa.

Announcement was made that the following candidates were elected by the Board of Direction, June 6th, 1899:

As Associate.

Edward James Farrell, New York City.

As Juniors.

Simon Novelo Cacères, New York City.
Richard Alexander Given, Tutwiler, Miss.
Isaac Harby, New York City.
Richard Willis How, New York City.
Reginald Horton Keays, Havana, Cuba.
Fred Dana Rhodes, New York City.

The Secretary announced the death of Robert Gillham, elected Member June 2d, 1886; died May 19th, 1899.

Adjourned.

THIRTY-FIRST ANNUAL CONVENTION, HELD AT CAPE MAY, N. J., JUNE 27th, 28th, 29th AND 30th, 1899.

First Session, Tuesday, June 27th, 1899.—The Society was called to order at 10 o'clock; President Desmond FitzGerald in the chair; Charles Warren Hunt, Secretary.

President FitzGerald delivered the annual address.

The President introduced Elmer L. Corthell, M. Am. Soc. C. E., who addressed the Society on the subject of the International Navigation Congress at Brussels in 1898.*

The President announced that a meeting of the Board of Direction would be held at the close of the present session.

At the President's request the Secretary made a few remarks in regard to the Society's work.†

The Secretary made supplementary announcements as to the Convention programme, and the special trip to Atlantic City, N. J.

Adjourned.

Second Session, Tuesday, June 27th, 1899.—The meeting was called to order at 15 o'clock; President FitzGerald in the chair; Charles Warren Hunt, Secretary.

The President announced the appointment of the following gentlemen as temporary chairmen of their several geographical districts for the purpose of the selection of members of the Nominating Committee:

District No. 1.—J. James R. Croes.
" " *2.*—Edward W. Howe.
" " *3.*—William R. Hill.
" " *4.*—Charles G. Darrach.
" " *5.*—Alfred Noble.
" " *6.*—Hunter McDonald.
" " 7.—D. W. Cunningham.

The Secretary made an announcement as to the Convention programme.

Discussion on the following subjects was called for by the President:

"Should the use of the method of Wheel-Concentrations be Discontinued in Determining the Stresses in Railroad Bridges?"

"In view of present knowledge of the Effect of Repeated Applications of Load, should Fatigue Formulas be used in Bridge Design?"

* For Mr. Corthell's remarks, see page 147.

† For this statement by Mr. Hunt, see page 153.

The Secretary read a written communication from J. P. Snow, M. Am. Soc. C. E., and the discussion was continued by the following:

Gustav Lindenthal, M. Am. Soc. C. E.
Palmer C. Ricketts, M. Am. Soc. C. E.
W. A. Doane, M. Am. Soc. C. E.
George S. Morison, M. Am. Soc. C. E.
W. H. Breithaupt, M. Am. Soc. C. E.
Walter L. Cowles, M. Am. Soc. C. E.
E. M. Scofield, Assoc. M. Am. Soc. C. E.
Henry B. Seaman, M. Am. Soc. C. E.

Adjourned.

Third Session.—At the close of the Business Meeting, Wednesday, June 28th, 1899.—President FitzGerald in the chair; Charles Warren Hunt, Secretary.

Discussion on the following subjects was taken up:

(*a*) "Should Stream Contamination by the Sewage of Cities be absolutely prohibited by law?"

(*b*) "Should the Purification of the Sewage of Cities be compulsory; and is this feasible for Large Cities?"

(*c*) "Is Filtration the coming solution of the Pure-Water Question for Cities?"

The following gentlemen presented their views:

R. E. McMath, M. Am. Soc. C. E.
W. C. Parmley, M. Am. Soc. C. E.
Gardner S. Williams, Assoc. M. Am. Soc. C. E.
Peter Milne, Assoc. Am. Soc. C. E.
Kenneth Allen, M. Am. Soc. C. E.
James Owen, M. Am. Soc. C. E.
Charles G. Darrach, M. Am. Soc. C. E.
E. W. Harrison, M. Am. Soc. C. E.
P. A. Maignen, Esq.

Discussions in writing from James H. Fuertes, M. Am. Soc. C. E., Edmund B. Weston, M. Am. Soc. C. E.; Louis L. Tribus, M. Am. Soc. C. E.; and A. McL. Hawks, Jun. Am. Soc. C. E., were announced by title.

Adjourned.

Fourth Session, Friday, June 30th, 1899.—President FitzGerald in the chair; Charles Warren Hunt, Secretary.

The Secretary read a telegram from Percival Roberts, Jr., President of the Pencoyd Iron Company, giving notice of special trains for those

wishing to visit Pencoyd on Saturday; also, a letter from the Vulcanite Cement Company, inviting the members to visit its works at Vulcanite, N. J., on July 3d.

The President called for discussion on the following subject:

"What are the economic conditions under which Electricity may be profitably substituted for Steam in the operation of Branch Railroad Lines, and what are the engineering requirements to be considered in such substitution?"

The following gentlemen presented their views:

H. S. Haines, M. Am. Soc. C. E.
H. G. Prout, M. Am. Soc. C. E.
Robert Moore, M. Am. Soc. C. E.
Oberlin Smith, M. Am. Soc. C. E.
W. B. Reed, Assoc. M. Am. Soc. C. E.
G. S. Davison, M. Am. Soc. C. E.
W. L. Webb, Assoc. M. Am. Soc. C. E.
J. James R. Croes, M. Am. Soc. C. E.
D. C. Jackson, M. Am. Soc. C. E.
W. J. Baldwin, M. Am. Soc. C. E.
Gustav Lindenthal, M. Am. Soc. C. E.
Edward Wegmann, M. Am. Soc. C. E.
C. W. Buchholz, M. Am. Soc. C. E.
A. P. Davis, Assoc. M. Am. Soc. C. E.

The following resolutions were passed:

Resolved, That the thanks of this Convention be extended to the officers of the Pennsylvania Railroad, and to the officers of the Philadelphia and Reading Railway for their great courtesy in extending to the members of this Society, so largely as they have done, the facilities of their lines in attending the Convention.

Resolved, That the thanks of this Convention be extended to the Local Committee for the most excellent arrangements made by them for the entertainment of the members and guests attending the Convention.

Adjourned.

BUSINESS MEETING.*

At the second session of the Convention, June 27th, 15 o'clock, the President announced the appointment of the following members as chairmen of the seven geographical districts into which the Society is divided for the purposes of the Nominating Committee (Art. VII, Sec. 2, of the Constitution):

District No. 1, J. James R. Croes; District No. 2, Edward W. Howe; District No. 3, William R. Hill; District No. 4, Charles G. Darrach; District No. 5, Alfred Noble; District No. 6, Hunter McDonald; District No. 7, D. W. Cunningham.

Wednesday, June 28th, 1899.—The meeting was called to order at 10.30 o'clock. President Desmond FitzGerald in the chair, Charles Warren Hunt, Secretary.

Nominations were received for members of the Nominating Committee from each of the seven geographical districts, and the following were elected members of the Committee for two years:

District No. 1.—Alfred W. Trotter,
" " *2.*—George B. Francis,
" " *3.*—T. McC. Leutzé,
" " *4.*—W. G. Wilkins,
" " *5.*—Augustus Mordecai,
" " *6.*—J. A. Ockerson,
" " 7.—Hiram N. Savage.

The Secretary presented a summary of the answers received relative to the time and place of holding the next Annual Convention,† and, after discussion, on motion duly seconded, the matter was referred to the Board of Direction.

The following resolution, offered by Samuel Whinery, M. Am. Soc. C. E., was adopted.

"*Resolved:* That the Board of Direction be requested to consider the advisability of urging upon the Superintendent of the Twelfth Census the importance of collecting and publishing with the next census, complete statistics of municipal works; and that the Board of Direction be authorized, if found desirable, to co-operate with the Census Department in formulating methods and preparing appropriate forms for the collection and publishing of such statistics."

The Secretary read a letter‡ from Mr. Percival Roberts, President of the Pencoyd Iron Works, inviting the members to visit their works, on Saturday, July 1st.

The following resolution, moved by Henry B. Seaman, M. Am. Soc. C. E., and amended by Past-President B. M. Harrod, was adopted:

"That the Board of Direction be requested to consider the propriety of providing for the payment of the expenses of the members

* For full report see p. 131. † See p. 134. ‡ See p. 139.

of the Nominating Committee in attending the meeting of that Committee; and to report with further suggestions in regard to methods for the nomination of officers to the Annual Meeting."

George S. Morison, Past-President, Am. Soc. C. E., moved that the matter of the revision of the Article of the Constitution relating to the nomination of officers be referred to the Board of Direction for consideration and report.

The motion was seconded and adopted.

J. F. O'Rourke, M. Am. Soc. C. E., asked whether any report had been received from the Committee on Standard Time, and the Secretary stated that no final report had been received from that Committee.

Mr. O'Rourke moved that the Board of Direction be requested to take up the matter of Standard Time and settle it.

The motion was seconded and adopted by a vote of 71 to 34.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

June 6th, 1899.—Vice-President North in the chair; Charles Warren Hunt, Secretary, and present, also, Messrs. Bensel, Clarke, Craighill, Morison, Owen, Ricketts, Thomson and Turner.

Reports of standing committees were received.

A progress report from the Committee on the Paris Exposition of 1900 was received.

The President was authorized to appoint a Committee to recommend the award of prizes for the year ending with the month of July, 1899.

The Secretary reported in regard to the matters connected with the Convention.

Applications were considered and other routine business transacted.

One candidate for Associate and six for Junior were elected.

Adjourned.

June 27th, 1899.—The Board met, as required by the Constitution, (Article VIII, Section 7), at the Stockton Hotel, Cape May, N. J., President FitzGerald in the chair; Charles Warren Hunt, Secretary, and present, also, Messrs. Bensel, Buchholz, Clarke, Harrod, Hering, Moore, Morison, North, Owen, Ricketts and Turner.

A report of progress was made by the Committee appointed to report on the Paris Exposition of 1900.

The Secretary announced the appointment by the President of the following members as a Committee to recommend the award of prizes for the year ending July, 1899.

THOMAS RODD, DWIGHT PORTER, JAMES L. LUSK.

The President was authorized to appoint a Committee to report to the Board recommending the time and place for holding the next Annual Convention not later than the October meeting.

The President was requested to appoint temporary chairmen of the several districts for the purposes of the Nominating Committee.

Adjourned, subject to the call of the President.

REPORT IN FULL OF THE BUSINESS MEETING HELD DURING THE THIRTY-FIRST ANNUAL CONVENTION, AT CAPE MAY, N. J., JUNE 27th and 28th, 1899.

Appointment of Provisional Chairmen.

At the second session of the Convention, June 27th, 15 o'clock, the President announced that in accordance with a vote of the Board of Direction he had appointed temporary chairman of the seven geographical districts into which the Society is divided for the purposes of the Nominating Committee; that it was expected that each of these Chairmen would call together the members from his district present at the Convention, and report their choice for member of the Nominating Committee at the Business Meeting to be held on June 28th at 10 o'clock. The appointments were announced as follows:

Chairman, District No. 1—J. James R. Croes,
" " " *2*—Edward W. Howe,
" " " *3*—William R. Hill,
" " " *4*—Charles G. Darrach,
" " " *5*—Alfred Noble,
" " " *6*—Hunter McDonald,
" " " 7—D. W. Cunningham.

Meeting Convened.

Wednesday, June 28th, 10.30 o'clock.—President FitzGerald in the Chair, Chas. Warren Hunt, Secretary.

The President.—Gentlemen, I am going to call this meeting to order, and I want you to unite with me in making it a success. The only way to accomplish this is to stop talking and have as little noise as possible, and I want you all to move up in front where you can hear what is going on. It is a great effort for any voice to be heard in this hall, and if we are to have a successful discussion this morning, and get through with the business, you must all unite to that end.

Election of Members of Nominating Committee.

The first business is that of the Nominating Committee; the reports of the temporary chairmen. I call on the Chairman of District No. 1.

J. James R. Croes, M. Am. Soc. C. E.—The members present from District No. 1 have directed me to present the name of Mr. Alfred W. Trotter as a member of the Nominating Committee.

The President.—Gentlemen, you hear the report of the Committee? What action do you take?

W. R. Hutton, M. Am. Soc. C. E.—We have, I think, the privilege of nominating another?

The President.—Certainly.

Mr. Hutton.—I offer in nomination Mr. C. C. Martin.

Geo. S. Morison, Past-President, Am. Soc. C. E.—I second the nomination.

The President.—What is the pleasure of the Convention? The Committee has nominated Mr. Alfred W. Trotter. Mr. Hutton has

Election of Members of Nominating Committee.

nominated Mr. C. C. Martin. The Chair awaits the motion. If there is no discussion, the Chair will put the question first on the report of the Committee. Those in favor of the election of Mr. Alfred W. Trotter will signify it by saying aye; those opposed, no. It is a vote.

District No. 2. The Chairman is Mr. Edward W. Howe. Mr. Howe is not present. Is there any report from that district?

Frank O. Whitney, M. Am. Soc. C. E.—The Committee met last evening and appointed Mr. Francis, of Boston.

The President.—One moment, please, Mr. Whitney. There is another suggestion that the Chair wishes to offer, and that is that every gentleman in getting up to speak give his name so that it may be taken by the reporter. Mr. Whitney.

Mr. Whitney.—The members of the second district met last evening and nominated Mr. Francis, of Boston.

The President.—The full name, please.

Mr. Whitney.—George B. Francis.

The President.—What is the pleasure of the meeting? Those in favor of the election of Mr. George B. Francis will signify it by saying aye; opposed, no. It is a vote.

District No. 3, Mr. William R. Hill, Chairman.

William R. Hill, M. Am. Soc. C. E.—Mr. President, the members from District No. 3 present the name of Mr. T. McC. Leutzé.

The President.—Gentlemen, you hear the report of Committee No. 3 presenting the name of Mr. T. McC. Leutzé. Those in favor of the election of this gentleman will signify it by saying aye; opposed, no. It is a vote.

No. 4, Mr. Darrach.

Charles G. Darrach, M. Am. Soc. C. E.—Mr. President, the members met and nominated Mr. W. G. Wilkins, of Pittsburg.

The President.—Gentlemen, you hear the report of Committee No. 4 nominating Mr. W. G. Wilkins. What is the pleasure of the meeting? Those in favor of Mr. Wilkins' nomination will signify it by saying aye; opposed, no. It is a vote.

District No. 5, Mr. Noble.

Alfred Noble, M. Am. Soc. C. E.—Mr. President, the members present from District No. 5 nominate Mr. Augustus Mordecai.

The President.—Gentlemen, you hear the report of the Committee from District No. 5 in favor of nominating Mr. Augustus Mordecai. Those in favor of his election will signify it by saying aye; opposed, no. It is a vote.

Mr. Hunter McDonald, Chairman of the Committee of District No. 6.

Hunter McDonald, M. Am. Soc. C. E.—The members from District No. 6 nominate Mr. J. A. Ockerson, of St. Louis.

The PRESIDENT.—Gentlemen, you hear the report of District No. 6 nominating Mr. J. A. Ockerson, of St. Louis. Those in favor of the election will signify it by saying aye; opposed, no. It is a vote.

District No. 7, Mr. Cunningham.

D. W. CUNNINGHAM, M. Am. Soc. C. E.—I had not been with any committee, and I do not know that any other member from District No. 7 is present.

The PRESIDENT.—Are you the only member?

Mr. CUNNINGHAM.—I am the only one from California. I do not know whether there is any from any other part of this district.

The PRESIDENT.—I do not believe the Convention will let you off, Mr. Cunningham.

Mr. CUNNINGHAM.—Perhaps you did not mean me. There is another Mr. Cunningham.

The PRESIDENT.—I meant you. You were nominated, you accepted the position and your name was posted.

Mr. CUNNINGHAM.—Well, I didn't know it.

A MEMBER.—You nominate the other man and that will let you off. (Laughter.)

The PRESIDENT.—Those in favor of the nomination of the other—— (Laughter.)

Mr. CUNNINGHAM.—I nominate Mr. Hiram Savage, of National City.

Mr. MORISON.—I think there are several members here from Texas which is in the same district.

The PRESIDENT.—There are. The report of the Committee, however, Mr. Morison, has been made and is now before the meeting. Those in favor of accepting the report of this Committee nominating Mr. Hiram Savage will signify it by saying aye. It is hardly necessary to put the noes. It is a vote. Well, those opposed, no.

There were no negative votes.

The PRESIDENT.—Gentlemen, I am happy to report that that part of the business is settled.

Mr. WILSON.—If you refer to Mr. A. C. Savage, of Mexico City, and I believe he is the gentleman referred to, I wish to correct his initials.

The PRESIDENT.—It is impossible to correct it now. If there are two Mr. Savages, the other Mr. Savage is elected. Mr. Cunningham, will you see if you made the correct report from your Committee? (Laughter.)

Mr. CUNNINGHAM.—I refer to Mr. Hiram Savage, of National City. I have forgotten his middle initial.

Time and Place of next Convention.

The PRESIDENT.—That is ancient history now. The next business, Mr. Secretary, the time and place for holding the Convention, which is always a bone of contention in this Society, will now be taken up.

The SECRETARY.—The Secretary has received the following votes:

VOTES RECEIVED IN REGARD TO TIME AND PLACE FOR HOLDING THE ANNUAL CONVENTION OF 1900.

Report on Time and Place of next Convention.

"Total number of votes received, 204.

"Distributed as follows:

Paris, France	41	Richmond, Va.	6
Washington, D. C.	15	Saratoga, N. Y.	4
St. Louis, Mo.	11	Pittsburg, Pa.	4
Boston, Mass.	10	Buffalo, N. Y.	4
Chicago, Ill.	8	Yellowstone Park, Wyo.	4
Cleveland, Ohio	7	Cincinnati, Ohio.	4
White Sulphur Springs, Va.	6	Duluth, Minn.	4
New York City.	6	Old Point Comfort, Va.	3
		Salt Lake City, Utah.	3

"Two votes for each of the following locations:

Detroit, Mich.	Denver, Colo.
Chattanooga, Tenn.	Nashville, Tenn.
Lake Champlain, N. Y.	St. Augustine, Fla.
San Francisco, Cal.	St. Paul, Minn.
Montreal, Que.	Alexandria Bay, N. Y.
Niagara Falls, N. Y.	Louisville, Ky.
Steamer to Duluth, Minn.	

"And one for each of the following locations:

Seattle, Wash.	Cape May, N. J.
Rochester, N. Y.	Omaha, Neb.
Milwaukee, Wis.	Lake George, N. Y.
Asheville, N. C.	Hotel Wentworth, N. H.
Topeka, Wash.	Los Angeles, Cal.
New Orleans, La.	Adirondacks, N. Y.
Albany, N. Y.	Houghton, Mich.
Toronto, Ont.	Texas.
Manitou, Colo.	Kansas City, Mo.
Mt. Desert, Me.	Chautauqua, N. Y.
Baltimore, Md.	Mackinac Island, Mich.
Scranton, Pa.	Mexico.
White Mts., N. H.	Great Lakes.
Ithaca, N. Y.	Providence, R. I.
Hotel Kaaterskill, N. Y.	Cresson, Pa.
Lake Mohawk, N. Y.	West Point, N. Y.
Saranac Lake, N. Y.	Portland, Ore.
South.	Havana, Cuba.
London, Eng.	Porto Rico.

"As to the time for holding the Convention the result is as follows:

March.	2	July.	42
April.	6	August	10
May.	14	September.	5
June.	95	October.	2

"Some of the above-mentioned votes were for the first part and last part of the month mentioned, and a few of them mentioned two consecutive months. Respectfully submitted.

"CHAS. WARREN HUNT,
"*Secretary.*"

R. B. Stanton, M. Am. Soc. C. E.—I think there was one point omitted there? Wasn't there one vote for the Philippine Islands? Discussion on Time and Place of next Convention.

The Secretary.—That was probably lost in the mails.

The President.—Gentlemen, you hear the report from the Secretary. What is your pleasure? The Chair awaits a motion.

Mr. Croes.—I move that the Convention be held in Porto Rico.

E. P. North, M. Am. Soc. C. E.—I second the motion.

Mr. Morison.—I offer an amendment. I would substitute Dawson City for Porto Rico.

Professor Fuertes rose to speak.

The President.—The gentleman did not give his name. (Laughter.)

E. A. Fuertes, M. Am. Soc. C. E.—I cannot give the name, but I will spell it: F-u-e-r-t-e-s.

The President.—Now, what is your motion, Mr. Fuertes?

Professor Fuertes.—I second the motion, sir.

Mr. Croes.—I believe that Professor Fuertes is the only native of Porto Rico present at the Convention.

The President.—Gentlemen, you hear the motion of Mr. Croes, seconded by Mr. Fuertes, that the next Convention be held at Porto Rico. Mr. Morison has the floor.

Mr. Morison.—I simply move to substitute Dawson City for Porto Rico.

Henry B. Seaman, M. Am. Soc. C. E.—(Spelling) S-e-a-m-a-n. I move to amend the motion by referring the matter to the Board of Direction.

The President.—That is an amendment to an amendment. Do you hear the motion, gentlemen?

A Member.—Mr. Morison's amendment was not seconded.

The President.—The Chair will put the amendment to the motion proposed by Mr. Seaman, as it is a motion to refer. Those in favor of referring this to the Board of Direction will signify it by saying aye; opposed, no.

The motion was carried.

The President.—That settles the place of the Convention. Are there any announcements to be made, Mr. Secretary?

The Secretary.—I might repeat the announcement as to the train on Thursday for Atlantic City. There may have been somebody who did not hear it. The train leaves here over the Pennsylvania Road from the Pennsylvania Depot at 8.30 o'clock sharp. Announcement as to Excursion.

Mr. North.—In the morning?

The Secretary.—8.30 o'clock, sir, is always in the morning, according to the nomenclature adopted by the Society. (Laughter and applause.) It will take about two hours to run over to Atlantic City and the train will probably return, leaving Atlantic City, about half-past

Announcement as to Excursion.

twenty—(laughter). Oh, no, no—(Laughter and applause)—half-past sixteen, and get here at half-past eighteen or a quarter to nineteen. (Laughter.)

JOHN F. O'ROURKE, M. Am. Soc. C. E.—Is half-past twenty ten o'clock?

The PRESIDENT.—Please address the Chair and give name.

The PRESIDENT.—Is there any business now before the meeting before we take up the regular discussion? The Chair awaits any motion.

Resolution as to Collection of Municipal Statistics.

SAMUEL WHINERY, M. Am. Soc. C. E.—I have a resolution to offer which I presume is appropriate now. It is this :

"*Resolved:* That the Board of Direction be requested to consider the advisability of urging upon the Superintendent of the Twelfth Census the importance of collecting and publishing with the next census, complete statistics of municipal works; and that the Board of Direction be authorized, if found desirable, to co-operate with the Census Department in formulating methods and preparing appropriate forms for the collection and publishing of such statistics."

The PRESIDENT.—Will you please pass that up here?

Mr. NORTH.—Mr. President, I wish to second that motion,

The PRESIDENT.—Gentlemen, you hear the motion of Mr. Whinery, seconded by Mr. North. The Chair will read the resolution. (The President read the resolution.) What is your pleasure in regard to this resolution?

Discussion on Collection of Municipal Statistics.

Mr. WHINERY.—Mr. President, as it is quite generally known, some effort has been made in the past two censuses of the United States to collect, collate and publish information about municipal and other public works. To a very large extent this has been a failure. I understand that in the last census a very large amount of information was collected, but it was found in such fragmentary shape and so incomplete that very little of it was published. Now I know of no class of information that is more interesting and valuable to the public, and certainly there is none that would be of more value to our profession than complete statistics and information on the extent and cost of the municipal works of American cities. Practically everything that has been done in collecting such information in this country has been by private effort. Some of our engineering publications have collected and published information in one or more branches of municipal work which has been valuable and useful.

The Census Department is collecting information, the most minute information, on very many other subjects of far less importance to the public, and certainly of far less importance to the profession of the civil engineer than these statistics about public works. For instance, any of us are far more interested in knowing the character, the extent and the cost of the sewerage systems of American cities than we are in knowing the number of sheep contained in any one State. It is useless to enlarge on this point.

I very much fear that unless some effort is made, the result in the coming census may be as it has been heretofore; that is, that there will be a failure to collect the necessary information and to publish the results, and it seems to me a very proper thing that this Society should take some action in the matter and should urge upon the Superintendent of the Census the importance of collecting in a complete and systematic way all this class of statistics, and collating, tabulating and publishing them in proper form. (Applause.)

Robert Moore, M. Am. Soc. C. E.—Mr. Chairman, I think all the members will agree that it is very desirable that statistics of this kind should be gathered. Whether the Census Department is the proper machine by which to do that is, I think, much more than doubtful. I think those who have examined our census reports, or the last ones, and I am giving the opinions of others than myself, the opinions of experts, have found that it has been loaded down with too many subjects; that the Census Department has undertaken a great deal more than it is possible for it to accomplish successfully, and that this is particularly true with lines of inquiry which call for special knowledge. You turn the ordinary census enumerator loose in gathering the statistics of municipal works, and he is certain to make a botch of it. And this is confirmed by the experience of the Department of Labor, which is to the effect that the only successful way of gathering statistics of the technical sort particularly, is to send skilled men who go to the places which they are to examine and spend time enough, by personal inquiry and by interrogating the proper persons, to gather the statistics. I think if you employ machinery which is organized very well perhaps for gathering the numerical statistics of population upon a problem of so entirely different and so largely technical a nature that you are certain to fail, and that it should be done in some other and better way, should be referred to some department, like the Department of Labor, which has machinery for gathering statistics of this special kind. So that, as the motion stands, I think I should be obliged to vote against it.

Mr. O'Rourke.—Mr. Chairman, I would like to ask Mr. Whinery whether it is not true that statistics of that description are collected by experts in the special branches.

The President.—Mr. Whinery will answer the question.

Mr. Whinery.—Mr. President, I presume they are, to a considerable extent, but those statistics are not accessible to the public or to the profession at large in many instances.

Mr. O'Rourke.—So that it is likely that the Census authorities would be able to collect as good information on that subject as anybody else, and then get the advantage of their machinery of publication and circulation.

Mr. Whinery.—Yes. Mr. President, if I may add right here in

Discussion on Collection of Municipal Statistics (continued).

reply to Mr. Moore—I intended to defer it until later, but as I am on my feet I may as well speak of it now. I think it would be impracticable to collect this information in that way. If collected in that way it would be unreliable and incomplete, but I think a moment's consideration will show that it is not necessary to resort to such means for collecting this information. It is not a matter of going around or asking this man how old he is or how many sheep he has or that sort of thing. It is the sending of some competent person to these cities, and I do not see why competent men should not be employed in that way. There are plenty of competent men who could be secured for that service. It is a matter of sending to these different cities a man to collect the information which is generally of record in these cities, so that the method of collection would be entirely different from that ordinarily employed in collecting information for the Census. So that I think that the objection which has been made probably lacks force.

The PRESIDENT.—I wish to give notice that at eleven o'clock, the Chair will call for another subject. You have five minutes.

Mr. CROES.—Mr. President, I hope that this resolution will pass in the form in which it was offered by Mr. Whinery. It is a resolution, as I understand it, requesting the Board of Direction to take this matter into consideration. There is now, as has been confessed by the other members present, no body or branch of the Government which is authorized to collect this information. The Census Bureau appears to be at present a body which will be most qualified to collect it. It is the business of the Board of Direction to inquire into that question and to see whether the Labor Department or the Census Bureau would be the most fitted for the purpose. Nobody can question the necessity and desirability of having such a collection of information made. Anybody who has been trying, as I have been, for twenty years past to collect statistics in regard to municipal works knows the utter impossibility of getting any reliable data from the country at large, and this would be a means of collecting; and we want the Board of Direction to inquire into the possibility and the feasibility and the best method of collecting the desired information. This is a definite motion referring to the Board of Direction the question, which is the best source of obtaining that information, and it puts the matter in the hands of a few men who can consider it and who will report to the Society, and if they find that the Census Bureau is the best, they need not report to the Society, and can go on and request the Superintendent of the Census to collect this necessary information in a rational manner, which has never been done by any department of the Government or any State Department, or any municipality or any Society at all. The only attempt that has been made to formulate the subject I believe has been by the new Water-Works Association which has gotten up a series of questions and a form of

publication of statistics of water-works in New England. Those questions need revising, and that information needs revising very much indeed, in order to make it what is now desired. I hope that the motion will pass.

Mr. Moore.—Will the Chair read the question once more?

The President.—Yes. I notice in this, Mr. Whinery, that you have not asked the Board of Direction to report or do anything but consider. It is possible a word is left out here: "Resolved, that the Board of Direction be requested to consider the advisability of urging upon the Superintendent of the Twelfth Census the importance," etc. Whether you mean to consider the——

Mr. Whinery.—Read the final clause, please, Mr. President.

The President.—"And if found desirable to co-operate with the Census Department"—Yes, I see; that is, you give the Board of Direction authority.

Mr. Whinery.—Yes.

Mr. Croes.—That is a very comprehensive resolution.

The President.—"To co-operate with the Census Department in formulating methods and preparing appropriate forms for the collection and publishing of such statistics."

Mr. Moore.—I see that it does not contemplate, however, calling on any other department than the Census Department, even if the Board should think best to do so.

The question was called for by several members.

The President.—The question will now be put. Those in favor of that resolution will signify it by saying aye; those opposed, no.

Vote on Collection of Municipal Statistics.

The resolution was adopted.

The President.—The Secretary will read a letter.

The Secretary.—I have a letter from Mr. Percival Roberts, President of the Pencoyd Iron Works, addressed to the Secretary and written June 26th, 1899, as follows:

"Philadelphia, June 26th, 1899.

"Mr. Chas. Warren Hunt,

"*Secretary, American Society of Civil Engineers,*

"Care of Stockton Hotel, Cape May, N. J.

Invitation from the Pencoyd Iron Works.

"Dear Sir,—It has been suggested to me that possibly some of the members of the Society may desire, while in this vicinity, to visit our works at Pencoyd. If such is the case we should be extremely happy to see any of them at their convenience, and will make the necessary arrangements for an examination of the plant.

"If it should be agreeable to any considerable number to fix upon Saturday next as the time, we should be very glad to arrange, probably with the Reading Railroad Company—if a sufficient number signify their willingness to be present—to have a special train run directly from the Terminal Station to Pencoyd and return, if we are advised, say, probably not later than Thursday next.

"I have requested Mr. Schneider to confer with you in the matter and make any necessary arrangements concerning the same."

Now, whatever arrangements are made for such a visit as Mr. Roberts outlines, will depend on the number of people who wish to go. If any one who desires to visit the Pencoyd Iron Works will report that fact in the office of the Secretary at the other end of the hall, the Secretary will report to Mr. Schneider, who assures me that they will be very well taken care of.

The PRESIDENT.—Has any gentleman any business of importance to bring up before the business meeting is brought to an end?

Resolution Relative to Expenses of Nominating Committee.

Mr. SEAMAN.—Mr. President, if new business is in order there is one subject which I would like to mention and possibly hear opinions from individuals, and if they see fit, a motion can be made referring the matter to the Board of Direction with the request or with power to act. It is the question of the expenses of the Nominating Committee in attending the meeting in August for the nominations. I was a member of District No. 1, and live in New York City, so that my expenses were only nominal; but when people come from California or St. Louis and have to pay their own expenses, it does not seem to me quite fair; and unless the Society makes some provision to bear the expenses which are incurred in its service, I think the result will be that we shall not have a representative attendance at these meetings. I therefore would suggest that a motion be made, if it meets the approval of the meeting, that the matter be referred to the Board of Direction with the request that the necessary expenses incurred in attending these meetings be borne by the Society.

The PRESIDENT.—Do you make a motion, Mr. Seaman, to that effect?

Mr. SEAMAN.—I will make a motion to that effect. Perhaps that will draw out the opinion of the members present. (Seconded.)

The PRESIDENT.—Will you please put it in writing, Mr. Seaman, and I will put it now.

Discussion on Expenses of Nominating Committee.

J. N. CHESTER, Assoc. M. Am. Soc. C. E.—As an ex-member of the Nominating Committee, having felt the importance of this, I second Mr. Seaman's motion.

The PRESIDENT.—You hear the motion, gentlemen. Those in favor——

PALMER C. RICKETTS, M. Am. Soc. C. E.—Is there to be no discussion of this motion now?

The PRESIDENT.—There is no time for discussion.

Mr. RICKETTS.—What I have to say will be very brief. Why should not this apply to the members of the Board of Direction also? Where are we going to stop if we begin with this? I think this matter ought to be pretty carefully considered. If the expenses of the members of the Nominating Committee, from California, for instance, are to be paid by the Society, why not pay the expenses of a member of the Board of Direction from California or New Orleans, and pay it once

a month. It would cost us somewhere between one and twenty thousand a year, I suppose. I think this question ought to be pretty carefully considered. If a man accepts such an office, I think he ought to be willing to stand some of this expense. We cannot tell where to stop in this matter.

Mr. Whinery.—This motion, as presented by Mr. Seaman, consists of two parts. The first is that the question be referred to the Board of Direction. The second is that the Board be recommended to bear these expenses. I am in favor of the first part of the motion, but I move to amend by striking out the latter part.

The President.—Has Mr. Seaman got his motion in writing yet?

Mr. Seaman.—I think the stenographer has it.

The President.—The stenographer will read the motion.

The stenographer read Mr. Seaman's motion as follows:

"I therefore would suggest that a motion be made, if it meets the approval of the meeting, that the matter be referred to the Board of Direction with the request that the necessary expenses incurred in attending these meetings be borne by the Society. * * * I will make a motion to that effect."

The President.—Is the motion in the form that you intended it, Mr. Seaman?

Mr. Seaman.—Yes, that is right. I do not wish to do more than my share of the talking on this subject, but as to the Society paying the expenses of the members of the Board of Direction in coming to their meetings I would suggest that the votes of the Board of Direction are often received by mail, and that votes sent by mail are ruled out at meetings of the Nominating Committee; that a man must attend the meeting of the Nominating Committee to have a vote. Furthermore, people do not accept positions on the Nominating Committee. They are nominated whether they attend here or not, and the vote of the meeting on their appointment is taken whether they attend here or not, and they are afterward notified.

Mr. Croes.—Mr. President, I would like to suggest that Mr. Seaman divide his motion into two parts; the first being as is stated, that the Board of Direction be requested to consider the propriety of paying the expenses of the members of the Nominating Committee, and the other being that the meeting recommends the adoption of a resolution by the Board to that effect or recommends that the Board of Direction favorably consider this proposition. That would bring out the sense of this meeting. It would refer the matter to the Board of Direction and bring it before the Board in such a way that they cannot avoid entertaining the question, and then it will be simply a question of the opinion of this meeting with reference to the propriety of it. Will Mr. Seaman divide his motion in that way? In the first place that the Board of Direction be requested to consider the propriety of paying the expenses of the members of the Nominating

Discussion on Expenses of Nominating Committee (continued).

Committee in attending the meeting at which the nominations are made. That would be the first proposition.

W. W. Curtis, M. Am. Soc. C. E.—I would like to ask if there is any provision in the Constitution or By-Laws of this Society whereby a member of the Nominating Committee unable to attend can decline and some one have the right to replace him? It seems to me that is a very necessary feature in this. We constantly elect people on that committee who are not present. If there is no provision whereby a gentleman who is elected and cannot attend cannot secure representation for his district, then it seems to me that the Society can fairly and reasonably make provision for expenses under such circumstances.

Mr. Croes.—That is a question of the Constitution I take it.

The President.—I think the Secretary may have a suggestion to make about that. I think there is a provision so that a quorum——

The Secretary.—I think the idea of making the Nominating Committee fourteen with the last five living Past Presidents of the Society (nineteen in all) was for the purpose of securing a representative meeting of the Nominating Committee. There are two members from each of the seven geographical districts, one holding over from last year, and then one from each of these districts elected this year. That is the only provision I know of that would cover the matter.

Benjamin M. Harrod, Past-President, Am. Soc. C. E.—My experience with the Nominating Committee is limited to one year, and I am not sure that I am familiar with its methods of proceeding, but it seems to me that part of this difficulty, if not all of it, could be met by giving to correspondents the right to ballot, and by prolonging the proceedings of nominating long enough to transmit to the members of the committee the proceedings of the first meeting, allowing them to write back and give their views, and give to those views the weight of a ballot, and I think if that goes before the Board of Direction they could outline some new method for the proceedings of the committee that would meet the case in point; simply to have a meeting of those who can attend at first, let the proceedings of that meeting be transmitted to all the members, let the other members who are not present join by correspondence and give that correspondence the weight of a ballot.

A Member.—A vote by letter ballot in other words?

Mr. Harrod.—By letter ballot.

Mr. Chester.—Another suggestion I think of, and that would be to change the date of the election of this Committee to the Annual Meeting in January, allow the organization of the committee to take place between that time and the Annual Convention; then let the meeting be held at the Annual Convention, and nominations take place at that time would be in ample time for discussion or change before

January next year. That is, elect the committee one year before the election of the officers instead of at the Annual Convention.

Mr. Morison.—Mr. President, I have served as a member of the Nominating Committee for two years longer than Major Harrod. I think there are two or three points in this connection which can be briefly stated and which show that we cannot for the present change the method of proceeding. I will say, as a preliminary matter to this, that I think our method of nominating officers can be very materially improved, but that it cannot be done without a change of the Constitution. I think the true method of nominating officers is by sending suggestions around to which replies, in the nature of letter ballots, will be received; but that cannot be done under the present provisions. The Nominating Committee, as it now exists, is a committee absolutely independent of the Board of Direction, which makes its own rules and acts under them, and is itself a close body, reserving its own secrets, and an entirely independent committee. When it makes its nominations, those nominations are before the Society, and any other ten members can make another nomination which will appear on the ticket in precisely the same way as the nomination made by the Nominating Committee. But the committee has to meet and attend to its work in time to go through all the necessary routine prescribed in the Constitution. The committee must first meet and make its nominations, then must ascertain whether those nominations are accepted; after which it reports in accordance with the provisions of the Constitution.

The President.—On the first day of October.

Mr. Morison.—On the first day of October. The time is pretty short, but so far the time has proved to be enough. All members of the Nominating Committee are expected to attend that meeting. They do not all come, but any member of the Nominating Committee who does not attend has the privilege of sending any suggestions in which he sees fit. It has been ruled that he cannot give a proxy, for there is no provision in the Constitution authorizing the voting of the members of this committee by proxy. The result is that at the first meeting of the Nominating Committee, all communications are read and nominations are made. Then time remains to report all these nominations to the members of the Committee who are not present and to secure their acceptance. The members who are not present cannot vote. They can express their opinions, and they are informed of the action of the Committee. But that is all that, under the present form of the Constitution, they are capable of doing. I think, myself, that this is one of the points of the Constitution which is susceptible of very great improvement, but that is not a question now before the meeting. As regards the payment of a mileage, or expenses, to the members attending that meeting, on the face of it, so far as they are concerned, it would be a very just thing to do, but a Society of this kind cannot

Discussion on Expenses of Nominating Committee (continued).

proceed entirely on the basis of justice. We are organized for the benefit of our profession, and we must expect to give of our time, our work and our money for the benefit of the Society, and this is one of the occasions when it seems to me that it is much safer to rely on the patriotism, so to speak, of the members, than to draw on the comparatively limited funds of the Society. (Applause.)

Mr. Harrod.—I would like to ask, if this committee is entirely independent of the Board of Direction, whether it has not authority to receive a ballot by proxy? Is there anything in the Constitution that prohibits that? It makes its own record. It governs itself. Is it not competent to make such a regulation as that; and in three months from the first of July to the first of October, isn't there time for that Committee to correspond with its absent members and still get through on the first of October?

Mr. Howe.—Having served as Secretary of the Nominating Committee one year I think I can give some light on the question asked by the last speaker. The time, as Mr. Morison has said, is exceedingly short. After this meeting the Secretary of that Committee is obliged to send out notices to nineteen members of that Committee and get replies as to whether they will attend that meeting. As has been voted by the Nominating Committee, the meeting will be held on the third Wednesday in August. After that meeting is held, my experience is, that there is barely time to get replies to those who are nominated for the different offices and get the report into the hands of the Board of Direction by the first of October. As to voting by proxy, there is one difficulty about that. An absent member can send in his vote for some one for one office who may be elected to another office; or he might be desirous, if his candidate for President was not nominated for President, that he should be nominated for Vice-President, and in the discussions in the meetings of the Committee it has always seemed wise to the Committee that there should be no voting by proxy, and as to the opinions of the absent members, I remember at the last meeting there were several letters from persons who did not attend the meetings and their wishes, particularly as regards the Directors, were complied with, and the Nominating Committee nominated persons whom the absent members suggested.

Mr. Seaman.—I am heartily in sympathy with the suggestion of Mr. Harrod that this whole matter be done by correspondence, and I think that a modification of the motion to refer to the Board of Direction would be a proper one. So far as the recollection of Mr. Howe as to how last year's Committee worked, that the suggestions were received by mail, my recollection is directly to the contrary—that we had two letters last year and in neither case were the candidates elected.

Mr. Howe.—In answer to Mr. Seaman—what I meant to say was

that the wishes of the absent members, as regards the Directors from their district, were complied with.

Mr. Morison.—The matter I want to refer to is exceedingly brief, but it shows the difficulty that we labor under. The Nominating Committee is required to report at a fixed date. The Nominating Committee is appointed at the Convention, which is held at a movable date. The Convention date may vary from April to August. Last year it was held late in July. This year it is held in June, and it throws difficulties in the way which must be considered in this connection. I merely mention that to show the kind of difficulties under which we now labor.

Mr. Croes.—It seems to me that we are wandering off into a discussion of the Constitution and not of the question at issue before this Convention. It is simply the motion of Mr. Seaman that the Board of Direction be requested to consider the propriety of paying the expenses of the members of the Nominating Committee in attending the meeting. That is the motion as I take it now. Isn't that so, Mr. Seaman?

The President.—That is correct.

Mr. Croes.—That is the only motion there is before the house and we are wandering off into a discussion of the Constitution.

Mr. Harrod.—I want to amend Mr. Seaman's resolution if it is exactly as Mr. Croes stated, "and that the Board of Direction be authorized to further consider the matter and make further suggestion in regard to methods for the nomination of officers."

The President.—Does Mr. Seaman accept that amendment?

Mr. Seaman.—I do.

Mr. North.—May I suggest that it would be well for the Board of Direction to be instructed to report at the Annual Meeting their plan?

The President.—Major Harrod, was your amendment to consider the whole question or to consider the propriety of a report?

Mr. Harrod.—To consider the whole question.

The President.—The question now before the house is this: Mr. Seaman moves that the Board of Direction be requested to consider the propriety of providing for the payment of the expenses of the members forming the Nominating Committee in attending the meeting.

Mr. Croes.—Attending what meeting?

The President.—Of the Nominating Committee.

Mr. Croes.—At what date?

The President.—No date mentioned,—and to report their recommendations at the Annual Meeting. Those in favor of the motion please signify it by saying aye; opposed, no.

Vote on Question of Expenses of Nominating Committee.

The motion was carried.

Mr. Morison.—I think, in furtherance of Mr. Harrod's suggestion, that it would be well to refer the Article to the Board of Direction, and I would move that the matter of the revision of the Article of the

Motion Relative to Nomination of Officers.

Constitution relating to the nomination of officers be referred to the Board of Direction. (Seconded.)

The PRESIDENT.—Gentlemen, you hear the motion of Mr. Morison that the Article of the Constitution referring to this matter of the Nominating Committee be referred to the Board of Direction for consideration and report. Those in favor of that motion will signify it by saying aye; opposed, no.

The motion was carried.

Committee on Standard Time.

Mr. O'ROURKE.—Before the question of sewage contamination is taken up, I would like to ask whether any report has been received from the Committee on Standard Time?

The PRESIDENT.—Will the Secretary please answer that question?

The SECRETARY.—No report has been received from the Committee on Uniform Standard Time; the status of the matter being that at the last Annual Meeting of the Society, a resolution was passed requesting that Committee to make a final report at this Convention. The Committee was informed of that action, but nothing further has been heard from it.

Mr. O'ROURKE.—Mr. Chairman, I would then move that the Board of Direction be requested to take up that matter and settle it so that we do not have the same thing staring us in the face at every meeting.

The PRESIDENT.—I am afraid this means an hour and a half debate. Gentlemen, is that motion seconded?

A member seconded the motion.

Vote on Question of Standard Time.

The PRESIDENT.—It is moved and seconded that the Board of Direction be requested to take up the matter of Standard Time and settle it for all time. Those in favor of that motion will signify it by saying aye; opposed, no.

The Chair was unable to decide whether the ayes or the noes prevailed, and the question was put a second and a third time with the same result. There were calls for a division.

The PRESIDENT.—The house will be polled. Those in favor of that motion will rise and stand until counted.

Seventy-one members rose.

The PRESIDENT.—Those opposed will rise and stand until counted.

Thirty-four members rose.

The PRESIDENT.—The Chair was deceived; there was lots of noise in that minority. (Laughter.) It is a vote.

Adjourned.

THE INTERNATIONAL NAVIGATION CONGRESS HELD AT BRUSSELS IN 1898.

Remarks by Elmer L. Corthell, M. Am. Soc. C. E.

Mr. President, as I have not been able to prepare a complete report on this subject, I must give you only a very desultory one, but I hope to secure your interest and your co-operation and assistance in future congresses.

Four years ago at the Hotel Pemberton, Boston, I presented to the Annual Convention of the Society a written report on the subject of the literary product of the International Engineering Congress at Chicago. This year I am to give, not, as I say, a written or complete report, but a very desultory report on the International Navigation Congress held at Brussels last year. In 1898, some time in March, I think, I was appointed a delegate, duly commissioned by Mr. Sherman, the Secretary of State, with the remark, especially emphasized, that the department had no money to pay the expenses (laughter), but he would like to have me attend and would also like to have me write a report. Consequently on my return last fall I began to write a report, but professional engagements and an absence of three months in Mexico have prevented my forwarding to Secretary Hay anything more than a very brief preliminary report. He very promptly sent it to Congress. It was printed, and copies are to be had. I brought a few with me and have them here in case any one would care to have one. But I wish to make a report that would have more usefulness than simply a brief *résumé* of what happened at Brussels. I am not a good French scholar, and you can imagine the amount of arduous work that I have done and still have to do in the midst of other business, to make a full and satisfactory—that is, satisfactory to me even—*résumé* of about a thousand pages in French. I am endeavoring to get it into such shape that engineers will be interested in it, and that it will be useful to our profession and to the important navigation interests of the United States. I was also appointed—not a representative—but was asked by the Secretary of this Society, at the request of Mr. Fteley, the President in 1898, to be at Brussels as a member of the Society. On account of the war with Spain I was the only representative of the United States Government or of engineering societies of this country. There were a few gentlemen there from the United States as individual members, or adherents, of the Congress.

I wish to give you a brief *résumé* of this Congress in order to convey a correct idea of it, so that you may appreciate its importance as an engineering and commercial organization. At first there were two kinds of Congresses held. One was a Maritime Congress and the other was an Inland Navigation Congress. The Maritime Congress was held

first in Paris in 1889, and again in London in 1893. The Inland Navigation Congress was held first at Brussels in 1885, at Vienna in 1886, at Frankfort in 1889, at Manchester in 1890, at Paris in 1892, and at the Hague in 1894. At the Hague it was decided that the two Congresses should be united under the name of the International Congress of Navigation, intending by that title to embrace all subjects of navigation, from the head waters of the rivers, to the light houses out at sea. It was intended to be held in Italy in 1896, but on account of some complications which that country was in at that time it was decided that it would not be best to hold the Congress there, and it went over to 1898 at Brussels. The scope of this Congress is intended to and generally does now embrace the following subjects: Canals, canalized rivers, ship canals, river improvements, entrance channels, ports and harbors, their construction, maintenance and operation, practical questions involving appliances, plant, methods of navigation, etc. Papers are written upon these subjects, they are discussed, and the questions are decided upon by the Congress itself. In organizing a Congress the method generally followed is for the Government of the country where the Congress is to be held to invite other Governments of the world to send their delegates. Invitations to the Congress at Brussels were sent out by the Minister of Foreign Affairs, corresponding to our Secretary of State. Invitations were extended to all foreign governments, and the Organizing Committee invited societies, associations, etc., to send delegates.

There were 121 government delegates representing 24 nations. Then there were delegates of municipalities, engineering and other technical societies, chambers of commerce, navigation associations and commercial and industrial organizations of various kinds interested in any way in the subject of navigation. Of these there were 410 delegates Twelve engineering societies were represented by regularly constituted delegates. There were 59 of these delegates. Twenty cities were represented as cities, with 34 delegates. There were 37 chambers of commerce with 56 delegates. There were 75 navigation and commercial organizations with 45 delegates. There were industrial and agricultural societies; manufacturers, like manufacturers of dredging machines; the John Cockerill Works at Seraing, and other similar works; steamship companies; ship canal companies; in fact the whole range of navigation was represented at that Congress. There were private members in addition to these, numbering 840 ; that is to say, not delegates of governments or municipalities. The total membership was 1 371. Of these 862 were from countries other than Belgium. There were in all 35 nationalities represented. The number of civil engineers was about 600.

Comparing this Congress with previous Congresses, there were at the Hague in 1894, 1 048 members with 20 nationalities represented. At

Paris, in 1892, there were 1 042 members with 22 nationalities represented. At Brussels, in 1885, which was the earliest Congress, there were 407 members with 13 countries represented. This last Congress at Brussels was held under the "high patronage" of His Majesty King Leopold, King of the Belgians, who took really an active interest in it. There were four honorary presidents; one was Prince Albert and the others were the ministers of three of the departments that have to do with such matters. One of these ministers, the Minister of Agriculture and Public Works, presided at many of the meetings. You will see from this that the Government itself took an active interest in the proceedings. There was a committee of patronage of very distinguished men from all over Belgium. There were two general presidents, one of whom was always present at the meetings, and they were men who held important positions in the Belgian Government. There were various committees which organized the Congress. In the preparation of this Congress there was a committee of organization which arranged the subjects of the papers and sent them out in the fall and early winter of 1897, inviting papers on the subjects. There were 71 regular papers sent in, of which 20 were in German, 6 in English, 4 in Dutch, 41 in French; 1 066 pages in all and 123 pages of plates. There were also presented 25 to 30 papers, which were either too late to be admitted or were on subjects not comprised in the regular list of questions. The regular papers were printed and sent to the members of the Congress in time to be read before the meeting took place in Brussels. The papers consequently were well known to the members before they came to Brussels. The report of which I spoke in the beginning of my remarks, and which I am preparing, will contain about 200 printed pages with about 115 plates, which will be bound with it, and I hope this will be published by Congress, as the preliminary report was, and distributed, so that members of this Society can have the use of it. I may add that between 600 and 700 members were present at the meetings. It was probably one of the largest Congresses in the way of personal attendance that has ever been gotten together on any subject connected with engineering and navigation. The Belgian Government provided two very suitable buildings for the holding of this Congress. As you know, most of the foreign governments have buildings that are suitable for such assemblies. Unfortunately we do not have in Washington at the seat of our Government, buildings suited for the holding of International Congresses, but you probably know that an effort is being made to obtain the necessary legislation and appropriation for building what is to be called the Temple of Justice, in Washington, which can be used for the holding of such Congresses as that at Brussels. Probably within a few years we will have such a building, and then we will be on a par

with other governments which already have those buildings for such purposes.

Now, I will simply analyze briefly the subjects which the Congress at Brussels had before it. There were five sections, divided into 1, 2, 3 or 4, sub-sections. The five sections were: 1st, Canalized Rivers; 2d, Inland Canals; 3d, Tidal Rivers and Ship Canals; 4th, Maritime Ports; 5th, Tolls, Dues, Port Charges, Warehouses, Transfer Sheds, Docks and all the paraphernalia of ports. The engineers who wrote the papers were, many of them, appointed by their governments to write them; the best men in the countries were delegated to write important and thorough papers on these subjects, which were very fully, almost profusely, illustrated with plans of docks, warehouses, transfer sheds, locks, movable dams, canal sections, dredges and everything relating to facilities for navigation. This Congress was conducted by holding two full meetings, one at the beginning and one at the end of the Congress, where all the members assembled, and then, like the American Association for the Advancement of Science, the Congress went into sectional meetings of the five sections. These meetings were continued during the whole week, and at the end of the week at the second general meeting the reports of these sections were read to the full Congress and a vote taken by the Congress upon the decisions which had previously been made by each of the sections. In other words, it is a business Congress. To my mind this feature is a very important one.

We have Congresses in the United States and they have them in other countries which are intended simply for speeches and remarks and dissertations, and that is the end of it. The reports are filed away, and some are printed and distributed. But the object of these navigation Congresses is to get at something useful, and consequently the entire trend of discussion in the sections on every subject was with that purpose in view; and after the discussion was carried through on each subject, there was a vote taken in that section upon the points that were raised on a motion in reference to the question submitted. That would be the decision of that section. Then afterward these sectional decisions were referred to the full congress and there a vote was taken upon them. So that what we get as the result of that Congress is of great utility, because it shows the consensus of opinion on many important matters affecting navigation, construction, maintenance and operation. I need not go into the subject of excursions. That was splendidly attended to by the people in Brussels. Special trains were furnished us, and 600 or 700 members, with some ladies visited Ostend, Bruges, Heyst, Antwerp and Seraing. King Leopold gave an evening reception in the Royal Palace, and everything was done to not only give us a useful congress, but also great pleasure. Everything was conveniently arranged. Each morning we found in

our boxes—every man had his box—a *résumé* of the previous day's work in print and with a programme of that day in full detail. If an excursion was projected the programme would give the information when you were to go to the train, and when you were to arrive here or there, and when you might return. It was one of the most complete organizations for comfort and for useful work that I think ever undertook such service.

While one object of these remarks is to give you a report of the Brussels Congress, another object is to enlist your interest first in the Congress that is to take place next year at Paris, and then in the Congress that is to take place in 1902 or later; 1902 is a long way ahead, but still there are matters that need to be done within the next two or three months, or six months at least. Let me state that at the Brussels Congress last year it was decided to form a permanent organization of the Congress. It has hitherto been held in a desultory sort of way. It will die out at the end of each Congress to be resuscitated again in a haphazard sort of way in the next two or three years. It is in the future to be a permanent organization like the International Association of Tests or like the Railway Congress, and all the rules, regulations and methods to organize and conduct this permanent Congress are now under advisement by the Bureau of the Brussels Congress, constituted for the purpose and to report to the Paris Congress. I happen to know about it as I am on that bureau; being one of the vice-presidents of that Congress, I was placed upon it, and I am in correspondence with the General Secretaries at Paris and Brussels. It is to be organized in a permanent way at the Congress to be held in Paris in 1900.

A wish was expressed by a large number of delegates at the Brussels Congress to hold the next Congress after Paris in the United States. They asked me if it would be possible. I replied, "Yes, if you will come it will, because I think the matter can be arranged in the United States." They asked me personally to see what could be done about it. Well, I have really done a great deal, and I want to tell you briefly what I have done. In the first place I knew it was necessary to consult the authorities at Washington, and I have had several consultations with Secretary Hay and Assistant Secretary Cridler in regard to it. I am not at liberty to give all the information about the matter, but may say it is under advisement by them. In the report I am to make to them in October, when I return from Europe this summer, I am to give them the information on which the matter can be decided and sent through the proper channels to Congress. It will require legislation from Congress on three points—first in order to authorize the President to invite the Congress at Paris next year, and thereafter the governments of the world. Second, the departments, as you know, have no money to spend on anything that is not legislated for by Congress; consequently it is necessary to have an appropriation to pay the

running expenses of this Congress, the entertainment of foreign delegates, or anything else that might arise by way of expense. And third, to designate the Government building in which to hold the Congress. On my way back recently from Mexico, I saw the leading men of the committees of the Western Waterways Association, and the Lake Carriers' Association, the Secretary of the Chamber of Commerce and other leading commercial men in St. Louis, the Secretary of the Board of Trade in Chicago. In New York I have seen the Secretary of the National Steam Navigation Association. They are all in favor of holding the Congress in the United States, and will all assist. This matter comes to Congress next winter if the plan is carried out which we have in mind, and it will require this legislation then. These commercial bodies are ready to assist, and you know their assistance is very powerful in such matters. I hope that the members of this Society interested in such subjects will, when the time comes, lend us their assistance also. (Applause.)

STATEMENT OF VARIOUS MATTERS OF SOCIETY INTEREST, AT THE 31st ANNUAL CONVENTION, JUNE 27th, 1899.

By Chas. Warren Hunt, Secretary, Am. Soc. C. E.

The information which the President asks me to present was contained in a letter which I addressed to him some time ago, and as it is mixed up with other matters, will, I am afraid, be somewhat disjointed.

Informal Discussions.—At several meetings of the Society informal discussions on subjects of general engineering interest have been introduced, the object being to stimulate discussion and thereby to make the meetings themselves, and the subsequent publications, of greater interest to the members. Informal discussions of this kind have been had on "Road-Building," "Building Laws" and "Dry Docks—Stone *versus* Wood." All of these have been participated in by many members, and great interest has been manifested in this kind of discussion. With the exception of that on "Building Laws," the matter brought out by these discussions has been published. The latter discussion was principally of local interest, and was not thought by the Publication Committee to be of sufficient value to the members at large to warrant its publication.

At this meeting this system of informal topical discussion is to be tried for the first time at a Convention.

Exemption from Taxation.—Another matter of interest is, that a claim, under the law that the Society is exempt from taxation, was made to the Tax Commissioners of the City of New York some time ago and was by them rejected. A suit against the City of New York was then instituted, and a decision has been handed down which holds that the Society is exempt, and declares the assessment made by the Tax Commissioners void. This decision has been accepted by the Corporation Counsel of the City of New York, and, consequently, the Society will in future be saved between $2 000 and $2 500 per annum. (Applause.)

Herbert Steward Library Fund.—By the will of the late Herbert Steward, Assoc. Am. Soc. C. E., $2 000 was bequeathed for the purpose of forming the "Herbert Steward Library Fund," the interest of which is to be used for the purchase of books for the library. Mr. Steward also gave to the Library such of his technical books as the Society desired.

Use of Auditorium.—The Auditorium of the Society has been found very useful in several special extra meetings during the year, and on one occasion the New York Electrical Society held a large meeting in it with great satisfaction to its members.

Junior Meetings.—Meetings for the mutual improvement of Juniors of the Society have also been started. Three have been held so far, and they have proved a decided success.

Changes in Publications.—The changes in the publications have already been stated in the *Proceedings*, but as some of the members present may not have read all that is printed there, it might be well to speak briefly of them. Hereafter all discussions as well as papers will be printed in *Proceedings* in order to give an opportunity for cross-discussion before the final publication in the *Transactions*.

Up to very recently it had been the custom to print in each number of *Proceedings* a list entitled "Additions to the Library and Museum." The form in which this was printed made it of practically no value to any one, and it has been discontinued, at least for the present. In its place a new list has been started, which has been received with much appreciation by members, and which gives the name, places of publication, and, wherever possible, the price of leading articles in the various branches of engineering, published during the month. Sixty odd periodicals are now reviewed in this list.

An arrangement has been made with the various scientific publishers, so far as we can find the publishers, by which copies of engineering books are now donated to the Library of the Society immediately upon their issue, and in each number of *Proceedings* under the heading "New Books of the Month," brief notices of all such additions to the Library are regularly printed.

Library Work.—The work of re-classifying and re-indexing the Library has gone on continuously and some account of this work may be of interest. The cards used are standard library cards, and they are all typewritten, it having been found more rapid and the result more legible and uniform than if they were written, as is usually done, by hand.

As far as classification goes, it did not appear in starting ours that any work of the kind had been formulated which would be of service. The scheme adopted is, in brief, to divide the Library into very general classes, such as Railroads, Water-Ways, Water-Supply, Sanitation, etc., and to subdivide each of these into the most natural divisions from the engineering point of view. For instance, "Railroads" is divided into History, Location, Construction, Equipment, Operation, Legal Documents, Company Reports, State Reports and Government Reports. "Water-ways" is divided simply into Canals, Rivers, Harbors, Lakes, Oceans. "Water-Supply" is divided into General, Water, Water-Works, Water-Power and Irrigation. "Sanitation" into Sewerage, Drainage, Ventilating and Heating, House-Drainage, Sewage Disposal, Garbage Disposal, Health and Disease, etc., etc. Each of these general divisions of the Library has its own separate classified index. There is also the usual author's index which

contains a card for every book in the library. There is also a special subject-index for bringing out the minor details, as in "Railroads"—locomotives, rails, frogs, switches, etc.

A person desirous of securing information on any general branch of railroad work can immediately turn to all the books in the library on that branch, say, for instance, on Railroad Equipment. If he knows that Mr. Smith has written a valuable report and does not know what railroad it was for or about, he can find that report in the Authors' Catalogue. If he wishes to know what there is in the library on a specific subject such as railroad water tanks or cinder pits, he can find it in the Subject-Index.

Three or four cards, on the average, have been found necessary for each title. More than 25 000 cards have already been written. It is hoped that before long we may be able to advertise the fact that any member, on application to the Secretary, may have a search made on any subject upon which he may desire to have information, and the present index is a necessary step toward that end.

EXCURSIONS, ETC., DURING THE THIRTY-FIRST ANNUAL CONVENTION.

The arrangements for the Convention were in the hands of the following Committees:

Committee of the Board of Direction.

C. C. Schneider, *Chairman.*

Henry G. Morse, Charles Warren Hunt.

Local Committee.

L. Y. Schermerhorn, *Chairman.*

A. Bonzano,	L. M. Haupt,
Joseph M. Wilson,	J. C. Trautwine, Jr.,
Samuel Rea,	Charles W. Raymond,
Joseph T. Richards,	George S. Webster,
Theodore Voorhees,	R. W. Lesley.

Members of the Society and members of their immediate families were furnished with the following transportation facilities:

By the courtesy of the Pennsylvania Railroad about 270 members and guests, from New York and from New England points, were provided with a special free train, which left New York at 1.40 P. M. on Monday, June 26th, and, making stops at Newark and Trenton, ran through to Cape May, arriving there at about 6.50 P. M.

Transportation between Pittsburg and Philadelphia, and between Washington and Philadelphia, and intermediate points, was furnished by the Pennsylvania Railroad, which also provided a second special train, from Philadelphia to Cape May. This train, carrying about 250 members and guests, left Philadelphia at 4.30 P. M. on Monday, June 26th, and arrived at Cape May at about 7.10 P. M.

On Thursday, June 29th, through the courtesy of the Pennsylvania Railroad, a party of about 280 members and guests made an excursion to Atlantic City, N. J., where they visited the new iron pier, Young's pier, the sewage disposal works, the water-works and other points of interest. The party returned to Cape May in the afternoon, arriving in time for dinner.

On Saturday, July 1st, return transportation from Cape May to Philadelphia was provided through the courtesy of the Philadelphia and Reading Railway. Two special trains left Cape May at 8 and 8.15 A. M., arriving in Camden two hours later. From Camden a party of about twenty-five took a special train to visit the works of the A. & P. Roberts Company at Pencoyd, Pa., by special invitation of Mr. Percival Roberts, President of the company.

The remainder of the party spent two hours in inspecting Cramp's shipyard and were then conveyed by a special train, provided by the Philadelphia and Reading Railway, to examine the Improvements in the Approaches of that railway in Pennsylvania Avenue.

Luncheon was provided by the Local Committee, at the Reading Terminal, the Broad Street Station and at the Hotel Hanover.

At 2.30 P. M. a special train, provided by the Philadelphia and Reading Railway, left the Reading Terminal and arrived in New York at 4.40 P. M.

In the afternoon, parties, conducted by various members of the Local Committee, visited the Parks, the Water-Works and other points of interest in Philadelphia.

THE ATTENDANCE AT THE THIRTY-FIRST ANNUAL CONVENTION.

The following 315 Members were in attendance:

Aiken, W. A .....Harrisburg, Pa.
Albree, Ralph.....Allegheny, Pa.
Alden, Charles A..... Steelton, Pa.
Allen, Kenneth .....Baltimore, Md.
Asserson, Peter C....Brooklyn, N. Y.
Atkinson, John B....Earlington, Ky.
Averill, Frank L..Washington, D. C.

Bacon, John W .....Danbury, Conn.
Baird, Howard C...Phœnixville, Pa.
Baker, Holland W. ...St. Louis, Mo.
Baldwin, Fred. H.....Bayonne, N. J.
Baldwin, William J...New York City
Ball, Charles B....Washington, D. C.
Bauer, J. L.....New York City
Bausher, C. I.....Harrisburg, Pa.
Belden, Edgar T.....New York City
Belknap, W. R.....Louisville, Ky.
Bensel, J. A .....New York City
Bent, C. C. F.....Philadelphia, Pa.
Berg, Walter G.South Bethlehem, Pa.
Bergengren, F.....Harrisburg, Pa.
Betts, E. E.....Chattanooga, Tenn.
Binion, Joshua .....New York City
Blakeley, George H...Paterson, N. J.
Boller, Alfred P.....New York City
Bolton, Channing M..... .Rio, Va.
Bonzano, A.....Philadelphia, Pa.
Bott, John B.....Greensburg, Pa.
Brackenridge, J. C..Brooklyn, N. Y.
Brackenridge, W. A.,
Niagara Falls, N. Y.
Braine, L. F .....Brooklyn, N. Y.
Breithaupt, W. H....New York City
Breuchaud, Jules.....Yonkers, N. Y.
Brinckerhoff, H. W...New York City
Brooks, Frederick.....Boston, Mass.
Buchholz, Charles W..New York City
Buck, L. L.....New York City
Buck, R. S.....New York City
Burgwyn, C. P. E....Richmond, Va.
Burnham, G. W....Philadelphia, Pa.
Bush, Edward W....Hartford, Conn.

Carlile, T. J .....Philadelphia, Pa.
Carter, W. J .....Cleveland, Ohio
Chapin, L. E ..... Canton, Ohio
Chester, J. N.....Pittsburg, Pa.
Christian, G. L.....New York City
Clapp, George H.....Pittsburg, Pa.
Clark, L. V., Jr....Philadelphia, Pa.
Clarke, T. C..... ....New York City
Clement, F. H . .....New York City
Codwise, Edward B..Kingston, N. Y.
Coffin, Amory .....New York City
Cohen, Fred. W.. ...Harrisburg, Pa.
Colby, B. H.....St. Louis, Mo.
Collingwood, Francis.Elizabeth, N. J.
Condron, T. L.....Chicago, Ill.
Connor, E. H .....St. Louis, Mo.
Coombs, Robert D., Jr..Steelton, Pa.
Corthell, A. B..... .Boston, Mass.
Corthell, E. L.....New York City
Cowles, Walter L.....Pittsburg, Pa.
Crabb, Thomas.....Newark, N. J.
Crandall, C. L..... Ithaca, N. Y.
Craven, Alfred.....New York City
Croes, J. J. R.....New York City
Cunningham, A. C Washington, D. C.
Cunningham, D. W.Los Angeles, Cal.
Curtis, W. W.....Chicago, Ill.

Dalrymple, F. W.Hornellsville, N. Y.
Darrach, Charles G.Philadelphia, Pa.
Davis, A. P .....Washington, D. C.
Davis, Charles.....Pittsburg, Pa.
Davison, George S.....Pittsburg, Pa.
Deans, John S.....Phœnixville, Pa.
Doane, W. A .....Meadville, Pa.
Dunham, H. F.....New York City
Easby, M. Ward... Philadelphia, Pa.
Edwards, J. H....East Berlin, Conn.
Ellis, John W.....Woonsocket, R. I.
Endicott, M. T ...Washington, D. C.

Furley, P. P.....Atlantic City, N. J.
FitzGerald, Desmond....Chicago, Ill.
Flad, Edward.....St. Louis, Mo.
Foster, T. J. .....New York City
Francis, George B.... Boston, Mass.
Freeman, F. L....Washington, D. C.
Fehr, H. R.....Easton, Pa.
Freyhold, Felix...Washington, D. C.
Frick, Walter.... .. Carbondale, Pa.
Fritz, John.....Bethlehem, Pa.
Fuertes, E. A.....Ithaca, N. Y.
Fuller, George W.....Louisville, Ky.
Fuller, W. B. ... . Albany, N. Y.
Furber, William C .Philadelphia, Pa.

Garrison, F. L.....Philadelphia, Pa.
Gifford, George E.....New York City
Goad, Charles E.....London, Eng.
Gowen, Charles S....Sing Sing, N. Y.
Gartensteig, Charles...New York City
Graves, Edwin D.....Hartford, Conn.
Gray, William.....New York City
Green, B. R. .....Washington, D. C.
Green, H. B.....Buffalo, N. Y.
Greene, E. A.....New York City
Greiner, J. E. .....Baltimore, Md.
Guthrie, Edward B....Buffalo, N. Y.

Haight, Stephen S....New York City
Haines, H. S.....New York City
Hains, Peter C.....Baltimore, Md.
Harlow, James H.....Pittsburg, Pa.
Haring, James S.....Somerset, Pa.
Harrington, F. E.....Easton, Pa.
Harris, C. M.....New York City
Harris, Van Alen..Middletown, N. Y.
Harrison, E. W....Jersey City, N. J.
Harrod, B. M. .....New Orleans, La.
Haskins, W. J.....New York City
Hatton, T. Chalkley. Wilmington, Del.
Haupt, L. M.....Philadelphia, Pa.
Haven, W. A.....Buffalo, N. Y.
Hawley, W. C. ...Atlantic City, N. J.
Hayford, John F..Washington, D. C.
Hazen, Allen .....New York City
Hering, Rudolph.....New York City
Hill, John W.....Cincinnati, Ohio
Hill, William R.....Syracuse, N. Y.
Hill, Edwin A.....Washington, D. C.
Hinckley, J. F. .....St. Louis, Mo.
Hoag, S. W., Jr.....New York City
Hodgdon, Frank W....Boston, Mass.
Hogue, Charles Jay..Little Rock, Ark.
Hopkins, Charles C.....Rome, N. Y.
Hopper, John J.....New York City
Horton, Horace E.....Chicago, Ill.
Howe, Edward W.....Boston, Mass.
Howe, M. G.....Houston, Tex.
Hubbell, Clarence W...Detroit, Mich.
Hughes, W. M..... Chicago, Ill.
Humphrey, R. L....Philadelphia, Pa.
Hunt, Chas. Warren...New York City
Hunt, Conway B...Washington, D. C.
Hutchinson, G. H.. ..Conneaut, Ohio
Hutton, N. H.....Baltimore, Md.
Hutton, William R....New York City
Hyde, A. Lincoln... .Cleveland, Ohio

Jackson, D. C.....Madison, Wis.
Jenkins, William D...Baltimore, Md.
Johnson, Lewis E. .....Steelton, Pa.
Johnston, Horace G...Aultman, Ohio
Jones, Washington..Philadelphia, Pa.

Karner, William J..... Chicago, Ill.
Keller, C. L.....Chicago, Ill
Kibbe, A. S.....Chester, Pa.
Knapp, Louis H.....Buffalo, N. Y.
Knowles, Morris.....Pittsburg, Pa.
Kenly, William L.....Baltimore, Md.
King, Paul S.....New York City
Kingsley, M. W.....Cleveland, Ohio

Lathbury, B. B.....Philadelphia, Pa.
Latrobe, Charles H....Baltimore, Md.
Leisen, Theodore A..Wilmington, Del.
Leonard, H. R.....Philadelphia, Pa.
Lesley, R. W..... Philadelphia, Pa.
Lewis, F. H.....Philadelphia, Pa.
Lewis, Sidney F....New Orleans, La.
Leutzé, Trevor McC....Albany, N. Y.
Lindenthal, Gustav...New York City
Linton, Harvey.....Altoona, Pa.
Long, E. McL.....Shelby, Ohio
Low, G. E.....Maplewood, N. J.
Loweth, C. F.....St. Paul, Minn.

Lowinson, Oscar.....New York City
Lucas, D. Jones....Philadelphia, Pa.
Ludlow, J. L.....Winston, N. C.

Maclay, W. W.....Glens Falls, N. Y.
McComb, D. E....Washington, D. C.
McDonald, Hunter. Nashville, Tenn.
MacGregor, R. A.....New York City
McMath, Robert E.....St. Louis, Mo.
McMinn, T. J.....New York City
Magor, H. B .....New York City
Maltby, F. B.....St. Louis, Mo.
Marden, Walter R.....Pittsburg, Pa.
Marindin, H. L....Washington, D. C.
Martin, C. C.....Brooklyn, N. Y.
Martin, Wisner B.....New York City
Mayer, Joseph.....New York City
Melvin, D. N....Linoleumville, N. Y.
Merriman, M...South Bethlehem, Pa.
Merriman, T .....Washington, D. C.
Meyer, Henry C.....New York City
Mordecai, Augustus..Cleveland, Ohio
Miller, J. Imbrie ...Bryn Mawr, Pa.
Milne, Peter.....New York City
Mills, Charles M....Philadelphia, Pa.
Modjeski, Ralph.....Chicago, Ill.
Moore, Robert.....St. Louis, Mo.
Morison, George S...New York City
Morrison, H.P., W.New Brighton, N.Y.
Morse, C. M.....Buffalo, N. Y.
Nelson, Alexander H...Pittsburg, Pa.
Nenmeyer, Robert E..Bethlehem, Pa.
Noble, Alfred.....Chicago, Ill.
North, Edward P.....New York City

Ockerson, J. A.....St. Louis, Mo.
Opdyke, S. B., Jr...Philadelphia, Pa.
O'Rourke, John F.....New York City
Osborn, Frank C. ...Cleveland, Ohio
Ostrander, John E...Amherst, Mass.
Owen, James.....Newark, N. J.
Oxholm, T. S., W.New Brighton, N.Y.

Packard, R. G.....New York City
Pardee, James T.....Cleveland, Ohio
Parmley, W. C.....Cleveland, Ohio
Parsons, George W.....Steelton, Pa.
Pegram, George H.....New York City
Perrilliat, Arsène..New Orleans, La.
Peverley, Ralph .....New York City
Phillips, Asa E....Washington, D. C.
Pierce, William T..... Boston, Mass.
Pitts, Thomas D.....New York City
Pratt, Mason D..... Steelton, Pa.
Pratt, William A. ..Wilmington, Del.
Pratt, William A....Philadelphia, Pa.
Prior, Charles H..Minneapolis, Minn.
Prout, H. G. .....New York City
Polk, William A.....New York City
Porter, J. M..... .....Easton, Pa.
Purdy, Corydon T.....New York City

Quinlan, G. A.....Houston, Tex.

Rafter, George W...Rochester, N. Y.
Ramsey, Joseph, Jr .. St. Louis, Mo.
Raymond, C. W....Philadelphia, Pa.
Rea, Samuel.....Philadelphia, Pa.
Reece, Benjamin. .Wilmington, Del.
Reed, W. Boardman...New York City
Reinholdt, K. O. P.....Pittsburg, Pa.
Rice, Walter P .....Cleveland, Ohio
Richards, Joseph T. Philadelphia, Pa.
Richardson, T. F.....Clinton, Mass.
Ricketts, Palmer C... ...Troy, N. Y.
Riser, Knud S.....Beaver Falls, Pa.
Russell, S. Bent. .....St. Louis, Mo.

Sabin, A. H...Long Island City, N. Y.
Sabin, A. T.....Fulton, Ky.
Savage, A. C.....Parral, Mex.
Scaife, W. L..... .Pittsburg, Pa.
Schermerhorn, L. Y. Philadelphia, Pa.
Schmidt, Max E .....New York City
Schneider, C. C..... Pencoyd, Pa.
Scofield, E. M.....Youngstown, Ohio
Scovill, E. T.....Cleveland, Ohio
Seaman, Henry B.....New York City
Searles, W. H.....New York City
Seddon, James A.....St. Louis, Mo.
Shankland, R. M.....Chicago, Ill.
Schaub, J. W.....Chicago, Ill.
Skinner, Frank W.....New York City
Smith, J. Waldo.....Paterson, N. J.
Smith, Oberlin .....Bridgeton, N. J.
Smith, T. Guilford.....Buffalo, N. Y.

Snyder, George D..Williamsport, Pa.
Spencer, John C.....Pittsburg, Pa.
Spielman, J. G.....Pittsburg, Pa.
Stanton, Robert B.....Sewickley, Pa.
Starling, William...New Orleans, La.
Stevens, F. S ..... Reading, Pa.
Strickler, G. B..... Rehoboth, Del.
Stuart, A. A.....New York City
Stuart, Francis L..Washington, D. C.
Swensson, Emil.....Pittsburg, Pa.

Taber, George A .....New York City
Tait, John G.....New York City
Taussig, Hubert P.....St. Louis, Mo.
Taylor, Charles F....Syracuse, N. Y.
Thacher, Edwin.....Paterson, N. J.
Thackray, George E..Johnstown, Pa.
Thayer, Russell... Philadelphia, Pa.
Thompson, B.....Greensboro, N. C.
Thompson, Ellis D..Philadelphia, Pa.
Thomson, G. H.....New York City
Thomson, John.....New York City
Throop, A. T.....Niagara Falls, N. Y.
Trautwine, J. C., Jr.Philadelphia, Pa.
Triest, W. G.....New York City
Trotter, Alfred W.....New York City
Trundle, Horatio H... Leesburg, Va.
Turner, E. K.....Boston, Mass.
Tuthill, Job....Grand Rapids, Mich.

Ulrich, Daniel.....New York City

Vickers, Thomas McE.Syracuse, N. Y.

Waddell, J. A. L....Kansas City, Mo.
Wagner, Joseph C Philadelphia, Pa.
Wagner, Samuel Tobias,
Philadelphia, Pa.
Wallace, John F.....Chicago, Ill.
Webb, Walter L....Philadelphia, Pa.
Webster, G. S .....Philadelphia, Pa.
Webster, William R.Philadelphia, Pa.
Wegmann, E..... Katonah, N. Y.
Whinery, S.....East Orange, N. J.
White, T. S .....Beaver Falls, Pa.
Whitney, Frank O.....Boston, Mass.
Whitney, N. O.....Madison, Wis.
Whittemore, W. F....Hoboken, N. J.
Wilgus, W. J..... New York City.
Wilkins, W. G.....Pittsburg, Pa.
Wiley, W. H.....New York City.
Williams, Gardner S... Ithaca, N. Y.
Wilson, H. M.....Washington, D. C.
Wilson, Joseph M..Philadelphia, Pa.
Wimmer, Sebastian...St. Mary's, Pa.
Winfree, P. B.....Bradford, Pa.
Wölfel, Paul L.....Pencoyd, Pa.
Wood, H. B.....Boston, Mass.
Woods, Henry D.West Newton, Mass.

The total registered attendance at the Convention was as follows :

Members of the Society in all grades	315
Guests, including ladies	320
Total	635

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society is open every day, except Sunday, from 9 to 22 o'clock.

MEETINGS.

Wednesday, September 6th, 1899, at 20.30 o'clock, a regular business meeting will be held. Ballots for membership will be canvassed, and two papers will be presented, viz., "Pile-Driving Formulas; Their Construction and Factors of Safety," by Charles H. Haswell, M. Am. Soc. C. E., and "The Groined Arch as a Covering for Reservoirs and Sand Filters: Its Strength and Volume," by Leonard Metcalf, Assoc. M. Am. Soc. C. E. These papers were printed in the May number of *Proceedings*.

Wednesday, September 20th, 1899, at 20.30 o'clock, a regular meeting will be held, at which will be presented a paper by Henry S. Prichard, M. Am. Soc. C. E., entitled, "Insufficient Provision for Counterstresses in Railroad Bridges," and a paper by C. W. Hudson, Assoc. M. Soc. C. E., entitled, "Comparison of Weights of a Three-Hinged and a Two-Hinged Spandrel-Braced Parabolic Arch." These papers are printed in this number of *Proceedings*.

INVITATION TO MEMBERS.

The American Institute of Mining Engineers has tendered, through its Secretary, a cordial invitation to the members of this Society to attend its next meeting which will be held in San Francisco, Cal., about September 25th, 1899. This invitation includes the privileges of special rates and trains which have been arranged for the trip. Mr. Theodore Dwight, Assistant Treasurer, Am. Inst. Mining Engrs., 99 John Street, N. Y., is in charge of these matters and should be communicated with by any member who desires to avail of the invitation.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(May 10th to August 7th, 1899.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Eng. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Eng., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Can.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Indicator*, Stevens Institute, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 30c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 35c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c
(14) *The Engineering Record*, New York City, 12c
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City, 35c.
(18) *Railway and Engineering Review*, Chicago, Ill.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England.
(22) *Iron and Coal Trades Review*, London, England.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gaslight Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Electrical Engineer*, New York City, 10c.
(28) *Industries and Iron*, London, England.
(29) *Journal*, Society of Arts, London, England.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des Ecôle Spéciales de Gand*, Brussels, Belgium.
(32) *Memoirs et Compt Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mecanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill.,, 10c.
(42) *Transactions*, Am. Inst. Elec. Eng., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 75c.
(45) *Mines and Minerals*, Scranton, Pa. 20c.
(46) *Scientific American*, New York City, 10c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift des Vereines Deutscher Ingenieure*, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Duesseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereines*, Vienna, Austria.
(54) *Den Tekniske Forenings Tidsskrift*, Copenhagen, Denmark.
(55) *Ingeniören*, Copenhagen, Denmark.
(56) *Teknisk Tidskrift*. Stockholm, Sweden.
(57) *Teknisk Ugeblad*. Christiana, Norway.
(58) *Proceedings*, Eng. Soc. W. Pa. 410 Penn Ave., Pittsburg, Pa. 50c.
(59) *Transactions*, Mining Institute of Scotland, London and Newcastle-upon-Tyne.
(60) *Bridges and Framed Structures*, 358 Dearborn St., Chicago, Ill., 30c.
(61) *Proceedings*, Western Railway Club. 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.

LIST OF ARTICLES.

Bridge.

Street Bridges Over Railroad Tracks in Buffalo. (14) May 13.
Some Hints on Bridge Designing. (4) April, 1899.
The Atbara River Bridge (Soudan). (15) May 26; (12) June 2; (11) June 9.
The Greenpoint Avenue Bridge, Brooklyn, N. Y. (14) May 27.
The Alexander III Bridge, Paris; Construction and Erection of Arch Ribs. (13) May 25.
The Morison Lift Bridge over the Chicago River. (15) June 2.
The Kaiser Wilhelm Bridge. (14) June 10.
The Manhattan Valley Viaduct, New York City. (13) June 8.
New Glasgow Bridge. (12) June 2.
Carlsberg Viaduct, Denmark. (12) July 14.
Replacing the Newark Draw. (20) July 27; (14) July 29.
Construction of the Kornhaus Bridge, Berne, Switzerland. (14) June 17.
The Forbes Street Bridge, Pittsburg, Pa. (14) July 15.
The Scherzer Rolling Lift Bridge over the Fort Point Channel, Boston. (15) August 4.
Cantilever Bridge at Lewiston, Idaho. (18) July 8.
Bridge Across the Indus on the Indian State Railway. (11) July 28.
Notes on Recent French Bridge Construction. (13) July 6.
A Stone Arch-Rib Railroad Bridge. (14) July 8.
A Curious Railway Trestle Bridge Failure, Nemadji River, Wis. (13) June 15.
The Atlantic City Steel Pier. (14) July 1.
Pile-Driving with the Water-Jet at Blackpool. (11) May 26.
The Architectural Element in Engineering Work. (14) June 3.
Paints for Iron. (12) July 14.
Tests of Girder Bridges. M. Möller. (51) July 5.
Les Nouveaux Ponts sur le Rhin, à Bonn et à Dusseldorf. Henry Martin. (33) May 20.
Note sur la Mise en Place des Nouveaux Tabliers Métalliques des Ponts sur le Ciron et le Dropt. (38) July, 1899.
Notes sur la Construction du Viaduc du Viaur (Ligne de Carmaux à Rodez). M. Théry. (43) First trimestre, 1899.
Le Pont J. F. Lepine. L. Biette. (43) First trimestre, 1899.

Electrical.

Telephone Relays or Repeaters. William J. Hammer. (27) June 3.
On Wireless Telegraphy. Prof. A. E. Dolbear. (27) June 10.
Pollak-Virag Machine Telegraph System. (27) July 15.
Oceanic Research and the Submarine Telegraph. (26) May 19.
Etheric Telegraphy. W. H. Preece. (29) May 5.
Construction and Reconstruction of Telegraph Lines. C. H. Bristol. (15). June 2.
Lighting Railway Carriages by Electricity. (12) July 14.
Estimate for a 1537-Light Municipal Electric Lighting Plant for San Francisco, Cal. (13) May 25.
Artificial Illumination for Ordinary Business and Domestic Purposes. Leon P. Lowe. (24) August 7.
Central Station of the Boston Electric Light Company. (27) May 27.
On a Method of Measuring the Frequency of Alternating Currents. Harrison W. Smith. (7) June, 1899.
Some Methods of Measuring High Potentials with Low Potential Instruments. Prof. Samuel J. Barnett. (27) June 17.
A Simple Device for Measuring the Frequency of an Alternating Current. Hubert V. Carpenter. (27) August 5.
Rotary Transformers and Storage Batteries as Related to Long-Distance Transmission. Prof. W. L. Robb. (27) June 10; (13) August 3.
Relation Between Armature and Line Currents in Polyphase Transmission. Prof. Henry S. Carhart. (27) June 17.
Transformation of Three-Phase to Two-Phase Currents. P. G. Watmough. (27) July 22.
Three-Phase Four-Wire Low-Tension Systems. A. J. Bowie, Jr. (27) June 17.
The Development and Operation of a Local Electrical Transmission System. W. S. Barstow. (13) June 1.
Single-Phase Distribution of Electric Current. Herbert A. Wagner. (13) June 1.
Transformer Tests. W. Elwell Goldsborough. (13) June 1.
Mechanical Stresses on Dynamo Coils. (27) May 27.
Resistance Detector. John C. Henry. (27) May 27.
Transformer Core Losses. W. Elwell Goldsborough. (27) May 20.
Special Types of Alternators. E. Jarvis Patten. (27) May 20.
A New Variable Ratio Transformer. (12) May 5.
Electrical Radiation. Arthur V. Abbott. (4) April, 1899.
The Limitations of Power Subdivision by Electric Motors in Manufacturing Establishments. (42) April, 1899.
Callendar's Recorder and Platinum Thermometer. (11) May 26.
Economical Transmission and Distribution of Electricity from a Distance. Horace Field Parshall. (11) June 16.
A Differential Method for Testing a Single Transformer. S. E. Johannesen. (27) July 29.
Arc Machine Regulation. H. Franklin Watts (27) July 29.
Some Notes on Underground Distribution of Two-Phase Currents in New York City. E. A. Leslie. (27) June 17.
Some Non-Integrating Electric Meters. Prof. William Edward Ayrton. (11) June 9.

Electrical—(Continued).

The Radiophone at the Electrical Exhibition. (46) May 27.
On a Possible and Rational Simplification of Electric Units. Prof. A. Blondel. (27) July 29.
Electric Contact Device for a Drawbridge. E. T. Birdsall. (27) July 22.
The Effect of Intense Cold on Magnetic Hysteresis. Arthur H. Ford. (27) August 5.
Three-Phase Currents in the Manufacture of Calcium Carbide in Italy. Cesare Rio. (27) August 5.
On the Oxidation of Carbon at Ordinary Temperature by Means of Atmospheric Oxygen with the Production of Electrical Energy. Willard E. Case. (27) July 22.
The Electrical Installation for the 1900 Paris Exposition. Georges Dary. (27) July 15.
Underground Electrical Construction. Louis A. Ferguson. (13) July 20.
A New Primary Battery. J. D. Darling. (3) July, 1899.
Methods of Electrical Transformation. James Swinburne. (11) June 9.
On the Luminosity of the Rare Earths When Heated in Vacuo by Means of Cathode Rays. A. A. Campbell Swinton. (28) June 9.
Electricity in Marine Work. S. Dana Greene. (10) July, 1899.
Operating Cost of Horse and Electric Delivery Wagons in New York City. (27) July 8; (19) July 22.
Electricity in Steel Works. (12) July 14.
Electricity as a Factor in Modern Development. Prof. R. B. Owens. (10) June, 1899.
The St. Helens Corporation Electricity Works and Tramways. (26) July 21.
Ashton Corporation Electricity Works and the Oldham, Ashton and Hyde Electric Tramway. (26) July 14.
Some Electrical Features of the Power Station of the Consolidated Traction Company of Pittsburg. (17) August, 1899.
Western Electric Station in Copenhagen. C. Heutzen. (55) April 29.
Short-Distance Electric Power Distribution. James Swinburne. (9) August, 1899.
La Fabrication des Fils pour Usages Électriques. (36) May 25.
Les Tramways Électriques d'Amiens. (36) May 25.
Utilisation des Chutes du Niagara. état Actuel des Installations Hydro-Électriques. (33) June 17.
Usine Électriques de la Compagnie Générale des Voitures à Paris. J. Prouteau. (36) June 25.
Distribution d'Energie par l'Electricité dans les Ateliers de Construction de Machines de la Allgemeine Elektricitaets Gesellschaft de Berlin. (36) July 10.
Méthode de M. Liebenow pour la Mésure de la Résistance d'Isolement d'une Batterie d'Accumulateurs. F. Drouin. (36) July 10.

Marine.

The New Royal Yacht *Victoria and Albert*. (11) May 12; (12) May 12.
Electric Yachts in Holland. W. de Gelde. (10) June, 1899.
H. M. S. *Hermes* and *Highflyer*. (11) July 7.
The Trials of H. M. S. *Hermes*. (11) June 9.
The New British Battleship *Vengeance*. (11) July 21.
French Armored Cruiser *Jeanne D'Arc*. (12) June 9.
Water Tube Boilers, S. S. *Orlando*. (12) June 9.
The Work of the Naval Repair Ship *Vulcan*. Gardiner C. Sims and W. S. Aldrich (9) July, 1899.
The New German First-Class Battleship *Kaiser Friederich III*. (19) May 27.
The Old Type of Monitor. William H. Wiley, M. Am. Soc. C. E. (20) May 18.
On the Launch of a Battleship. H. R. Champness. (11) July 28; (12) July 28.
The Launch of H. M. S. *Vengeance*. (12) July 28.
On the Boiler Arrangement of Certain Recent Cruisers. F. T. Marshall. (12) July 28.
On Elswick Cruisers. Philip Watts. (12) July 28.
Express Boilers in War Vessels. (12) May 26.
Triple Expansion Engines. H. M. S. *Cheerful* and *Mermaid*. (12) July 21.
Swift Passenger Steamers of Moderate Size. Edwin Lewis Martin. (11) June 9; (12) June 9.
Merchant Steamers and War Requirements. John Harvard Biles. (11) June 9: (12) June 9.
Recent Developments in Cargo Steamers. Edwin William De Rusett. (12) June 9; (11) June 16.
S. S. *Kinfauns Castle*. (12) May 19.
Twin-Screw S. S. *Moskva*, for the Russian Volunteer Fleet. (11) May 12.
Engines of the Russian Volunteer Steamer *Moskva*. (11) May 26.
The Water-Tube Boiler in Passenger Steamers. (12) May 26.
Experiments on Thrust Block Friction. F. von Kodolitsch. (11) July 28.
Fighting the Ice in River and Lake. George E. Walsh. (10) June, 1899.
Ice-Breakers. H. F. Swan. (12) July 21.
Bucket-Dredging Machine. (11) June 25.
Graving-Dock Blocks and Appliances. Arthur John Maginnis. (11) June 23.
A New Thames Graving Dock. (12) July 28.
New Graving Dock at Troon. (12) July 14.
The Failure of a Timber Dock. (14) July 29.
The New Dry Dock for the U. S. Navy at Boston, Mass. (13) July 20.
Lock Entrances and Graving Docks. Robert Cope Hardy Davison. (11) June 9; (12) June 9.

Marine—(Continued).

The New Government Testing Tank for Ship Models at the Washington Navy Yard. D. W. Taylor, Naval Constructor, U. S. N. (13) July 6.
The Supporting of Ships in Dry Docks. Francis Elgar. (11) July 28.
Keyham Dockyard Extension Works. Whately Eliot. (11) July 28.
The Rise and Fall of the American Merchant Marine. Joseph R. Oldham. (1) May, 1899.
The Application of Mechanical Devices to the Riveting and Calking of Ships. Robert Twentyman Napier. (11) June 23.
Portable Pneumatic Riveters in Shipbuilding. (11) July 21.
The Discharging and Reloading of Large Steamers. William Henry Hunter. (11) June 9; (12) June 9.
Electric Power for Marine Propulsion. Charles T. Child. (9) June, 1899.
The Bending Strains in Z-Bars, used in Shipbuilding. A. Meyerhof. (48) May 27.
Dry Dock at Talcahuano, Chili. v. Horn. (53) May 19.

Mechanical.

The Use, Design and Manufacture of Iron in Ornament. Chester B. Albree. (48) March, 1899.
Tilting Open-Hearth Furnaces. Archibald P. Head. (11) May 12; (28) July 14; (50) June 1.
The Use of Blast Furnace and Coke-Oven Gases. Enrique Disdier. (28) May 12; (47) May 27.
The Use of Hot-Blast in the Bessemer Process. J. Wiborgh. (28) May 19; (11) July 14.
The Design of Hot-Blast Coils. (14) June 3.
Utilization of Blast-Furnace Gas. F. Zeyringer. (50) July 15.
The Use of Blast-Furnace Gas in Gas Motors and Tests of a 60-Horse-Power Gas Engine. E. Meyer. (48) April 29.
Use of Blast and Coke Furnace Gases. F. W. Lürmann. (50) June 1.
Smelting Furnace "Deutscher Kaiser" in Bruckhausen am Rhein. (50) June 15.
The World's Consumption of Pig Iron in the Steel Industry. J. S. Jeans. (20) June 1.
Steel Manufacture Direct from the Ore. Dimitris Tschernoff. (16) June 3.
Handling Materials in a Foundry. W. J. Keep. (20) May 25.
The Wellman Open-Hearth Furnace. (20) May 25.
Forgings and the Machines Used in Producing Them. (12) June 9.
The Crystalline Structure of Metals. J. A. Ewing and W. Rosenhain. (28) June 9.
Goldschmidt's Method of Measuring High Temperatures. F. Wüst. (50) July 15.
Crystallization and Segregation of Cast-Iron. P. Kreuzpointer. (13) June 1.
The Heyl & Patterson Pig Iron Casting Machine. (20) May 18.
The Influence of the Size of Test Piece on the Compressive Strength of Cast-Iron. (13) May 18.
Importance of Fillets and Round Corners of Modern Machinery Castings. John M. Richardson. (28) July 7.
Theories and Facts Relating to Cast-Iron and Steel. Bertrand S. Summers. (41) July, 1899.
The Elastic Properties of Steel Wire. Archer Dave Keigwin. (63) Part II, 1899.
Ductility and Working Qualities of Steel Castings. C Bach. (48) June 10.
The Influence of Casting Temperature Upon Steel Robert Abbott Hadfield. (11) June 23.
Pipe Flanges and their Bolts. A. F. Nagle. (24) May 15
Rolling Mill Fly-Wheels. John Fritz, M. Am. Soc. C. E. (20) May 18; (10) June, 1899.
Some Historic but Successful Fly-Wheels. (14) May 20.
The "Parole" Water Tube Boiler. (11) July 21.
A New Method of Forced Draft. (11) July 21.
Some Experiments Having Reference to the Durability of Water-Tube Boilers. A. F. Yarrow. (12) July 21.
Lyall's Water-Tube Boiler. (12) May 19.
Automatic Stoker at Messrs Charrington's Brewery. (11) May 26.
Investigations of Boiler Explosions. (14) May 20.
Efficiencies of Modern Steam Boilers and Furnaces. R. S. Hale. (9) July, 1899.
Experiments on the Heat-Absorptive Power of Water. (12) May 12.
A New Type of Compound Engine. (27) May 27.
Quick-Revolution Steam Engines. Arthur Herschmann. (48) April, 1899.
High-Speed Engines. John Handsley Dales. (63) Part II, 1899.
Wigzell's Triple Expansion Engine. (11) May 5.
Development of the Steam Turbine C. A. Parsons. (10) July, 1899.
Double-Cylinder Steam Fire Engine, London County Council. (19) May 20.
Vertical Steam Engines. G. Marx. (48) May 13.
A German Blowing Engine. (16) July 22.
High-Pressure Cornish Engine, Basset Mines, Cornwall. (12) May 12.
The Valves of a Watertown Four-Valve Engine. (14) June 10.
A New System of Valves for Steam Engines, Air Engines and Compressers. Fred W. Gordon (13) June 15; (20) July 13.
Some Uses of the Steam Engine Indicator. Elward F. Miller. (15) May 26.
On the Connecting-Rod Problem. Prof. Dunkerley. (11) June 2.
Motors for Driving a Train of Rollers. C. Kiesselbach. (50) May 1.
Oil Motors. E. Loos (53) June 16.
The Importance of High Pressure in Fuel Oil Burning. (25) June, 1899.

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Diesel's Rational Heat Motor. E. D. Meier. (7) June, 1899.
The Diesel Motor. (18) July 8.
Evolution and Future of Benzine and Petroleum Engines. G. Lieckfeld. (9) August, 1899.
The Monarch Launch Motor. (28) June 16.
Pneumatic Pyrometer. Uehling and Steinbart. (50) May 1.
The "Mammert" Pump. (55) July 1.
Test of a Pumping Engine at Chicago, Ill. (13) August 3.
Compound and Triple Expansion Pumping Engines. (14) July 8.
Condensers and Cooling Towers. (25) August, 1899.
The Efficiency and Design of Surface Condensers. Thomas Ernest Stanton. (63) Part II, 1899.
The Theoretical Treatment of Air-Cooling Water Towers. William A. Mosscrop. (13) July 13.
Separate Condensing Plants for Factory Purposes. Sir Alfred Seale Haslam. (11) June 23.
A New Gas-Generating Machine. (46) May 27.
Labor-Saving Appliances in Gas Works. Charles Claude Carpenter. (11) June 23.
Residual Products from Gas Purification. William Foulis. (11) June 23.
The Utilization of Coke Oven Gases for Lighting. (16) July 29.
Self-Intensive Gas Liquefier. (12) May 12.
The Products of Combustion of Illuminating Gas. (24) May 15.
Incandescent Mantles. Vivian B. Lewes. (24) May 22.
The Kern Incandescent Gas Burner. (24) May 29.
Some Special Features in the Connecting Mains of a Provincial Gas Company's Stations. W. O. Gibb. (24) May 22.
The Evolution of the Hydraulic Main. William Carr. (24) July 31.
A New Design in Gas Engines. (12) May 12.
Gas Engines as Motive Power in Engineering Works. A. R. Bellamy. (13) July 27.
The Evolution and Future of the Gas Engine. George Lieckfeld. (9) July, 1899.
Gas Tram Cars, the Black Pool, St. Anne's and Lytham Tramways. (12) July 28.
Hydraulic Transmission and Distribution of Power. E. B. Ellington. (9) June, 1899.
Mechanical Applications of Compressed Air. W. L. Saunders, M. Am. Soc. C. E. (3) July, 1899.
Compound Two-Stage Air Compresser. (20) May 25.
Four-Stage Air Compresser for Street Car Service, (13) June 8; (14) June 10.
The "Tauzin" Voiturette. (28) July 28.
The Blake Motor-Carriage. (28) July 21.
Automobolism in Paris. (46) May 13.
Electric Hacks at Paris. (19) May 27.
Motor Cab Trials in Paris. (12) June 16.
Delivery Van and Omnibus. Charles Mildé & Co. (26) May 19.
An Electric Fire Wagon. (12) July 7.
A New Automobile Fire Engine. (19) August 5.
Cranes and the Power to be used with them. Arthur Pitt. (11) June 23.
An Electrically-Operated 150-Ton Revolving Derrick. (47) May 27.
25-Ton Electrical Goliath Crane. (11) June 9.
The Brown Cantilever Traveling Cranes at the Cramp Shipyard. (20) June 22.
Coal Dock Plant of the Baltimore & Ohio Railway, Sandusky, Ohio. (18) July 1.
Temperley Transporters for Coal Handling. C. V. Slomann. (55) May 13.
The Production of Seamless Tubing. H. S. Wilson. (1) June, 1899.
Strength of Bicycle Tubes. Oberlin Smith, M. Am. Soc. C. E. (20) August 3.
The Folding Military Bicycle. (19) May 20.
The Influence of Motive Power on the Design of Cotton Mills. Stephen Greene. (10) July, 1899.
The Limitations of Power Subdivision by Electric Motors in Manufacturing Establishments. (42) April, 1899.
The Utilization of Waste. John Birkinbine. (2) July, 1899.
A New Type of Steam Road Roller. (13) June 1.
New Method for the Production of High Temperatures, and for the Preparation of Difficultly Fusible Metals, free from Carbon. M. Goldschmidt. (19) May 20.
Steam Hydraulic Press. 10 Million Kilogram Capacity. (50) July 1.
On Pneumatic Tools. Haedicke. (50) July 1.
Machine Tools. Arthur Greenwood. (11) June 23.
Pneumatic Propulsion. (12) June 2.
The Calculation of Centrifugal Ventilating Fans. (14) August 5.
Tests of Rubber Packing Rings at the State Laboratory. (55) April 15.
The Hastings Tunneling Shield. (13) August 3.
A Plant for the Commercial Manufacture of Liquid Air. (13) June 8.
Etude sur divers Gaz combustibles utilisés pour divers Usages industriels en général, et principalement pour la Production de la Force Motrice. A Lencauchez. (32) May, 1899.
Théorie Mathématique de la Machine à Vapeur. M. Nadal. (37) June, 1899.
Les Régulateurs des Machines à Vapeur. L. Lecornu. (37) April, 1889.
La Théorie Mathématique de la Machine à Vapeur. M. Anspach. (37) April, 1899.
Note sur la Chocs causés par l'eau dans les Conduites de Vapeur et sur les Ruptures de Valves en Fonte. M. Walckanaer. (43) First trimestre, 1899.
Bulletin des Accidents d'Appareils á Vapeur survenus pendant l'anné, 1897. (43) First trimestre, 1899.

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Déformations Permanentes et Rupture des Métaux. G. Faurie. (37) June, 1899.
Le Grande Sidérostat de 1900. P. Gautier. (42) May, 1899.
Machine à Additionner les Fractions. (36) June 10.
D'Éteudeur Régulateur de Pression et Hydro-Alimenteur Automatique pour Chaudiéres Systèmes Sandillon. (36) June 10
Nouvelles Voiturettes Automobiles à suspension Élastique. (36) June 10.
Deuxième Concours des Fiacres Automobiles. (33) June 24.
Laminage des Demi-produits, Rails, Traverses et Poutrelles en acier. Emile Demenge. (33) May 13.

Military.

The Toulon Explosion. Col. J. T. Bucknill. (11) May 26.
Krupp Quick-Firing Field-Gun Carriages. (12) May 26.
The Mine Defence of Santiago Harbor. Lieut. E. E. Capehart, U. S. N. (10) July, 1899.
The Story of Smokeless Powder. Hudson Maxim. (10) July, 1899.
Wave Action in Guns. Frederick H. McGahie. (19) May 27.
The Terrain. Lieut. C. W. Castle. (44) July, 1899.
Electric Communication in the Field. Howard A. Giddings. (44) July, 1899.
Some Properties of Cordite. Sir William Anderson. (63) Part II, 1899.

Mining.

Notes on Gold Mining. Fearnside Irvine. (11) June 16.
Modern Practice in Gold Mining. John Hays Hammond. (11) June 16
The Rose Deep Gold Mine. Johannesburg. (12) June 16.
Hydraulic Gold Mining. John Robert Richardson. (63) Part II, 1899.
Manganese Ore in Brazil. F. Greven. (50) May 1.
A New Center for the Russian Iron Industry. M. Glassenapp. (52) No. 7.
Feldspars and Kaolins of Southeastern Pennsylvania. Prof. T. C. Hopkins. (3) July, 1899.
The Iron Industry of Sweden David A. Louis. (9) June, 1899.
The Great Magnetite Deposits of Swedish Lapland. David A. Louis. (9) July, 1899.
By-Product Coke Ovens. Evence Coppee. (11) June 23.
Comparative Advantages of Electricity, Steam and Compressed Air for Mining Purposes. Henry Slade Childs. (11) June 16; William Edward Garforth. (11) June 23.
Problems in the Treatment of Butte Ores. A. H. Wethey. (16) July 1.
Testing and Sampling Placer Deposits. Edmund B. Kirby. (16) July 29.
Electricity in Coal Mining. John Price Jackson and Frank F. Thompson. (28) July 28; (19) August 5.
Ignition of Firedamp and Coal Dust by Means of Electricity. A Report of Experiments to Ascertain the Danger of Using Electric Appliances in Mines. (45) August, 1899.
Timbering Underground Workings. George L. Kerr. (45) June, 1899.
Mining and Ventilating Anthracite Mines with Increasing Depths. H. O. Prytherch. (45) June, 1899.
Current Ventilation; the Early Methods, Single Current Coursing, Splitting, and the Causes From Which They Originated. (45) August, 1899.
Life Saving in Mines. H. Rossner. (45) June, 1899.
Modern Mine Haulage Practice. Harry K. Myers. (2) July, 1899.
Modern Improvements in Coal Mining; Winding from Deep Mines. Henry William Martin. (11) June 16.
Locomotives de Mines à Vapeur et à l'Electricité. (36) June 25.

Municipal.

Good Roads in New York State. (14) May 20.
County Road Construction in Ireland. (14) May 13.
Concrete for Street Work. (14) June 3.
Subways for Street Pipes. (14) June 3.
The Development of Roads and Street Pavements. Francis W. Blackford. (1) April, 1899.
Road Making in Ontario. (14) June 10.
Public Works in Kansas City. (14) June 17.
Experience with Wood Pavements at Ipswich, England. (14) July 1.
The Use of Concrete Curbs and Gutters. (13) July 6.
The Strand Improvement, London. (12) July 7.
Redwood Block Paving with Asphalt Carpeting at Oakland, Cal. (13) July 20.
State Road Construction, New York. (14) August 5.
Notes on Indianapolis Pavements. (14) August 5.
New York Street Widenings and Extensions. (19) July 29.
Construction of Substructures in Streets. Henry C. Hodgkins. (24) July 24.
La Construction et l'Entretien des Routes. Cyrille Jansens. (30) June, 1899.

Railroad.

The Central of Georgia Terminals. (40) May 26.
The Brooklyn Terminal Improvements and East River Tunnel of the Long Island Railroad. (13) May 18.
Southern Pacific Terminal at Oakland. (15) July 21.
The Boston Elevated Railway. (13) May 11.
Building the Boston Elevated Railroad. Edgar M. Smith. (15) July 7.

Railroad—(Continued).

The Power Station of the Northwestern Elevated Railroad Company of Chicago. (17) July, 1899.
Gasoline Engine Drainage Plant at Sixteenth Street Crossing, Chicago. (18) May 13; (40) May 12.
New Gravity Yard, Illinois Central Railroad at New Orleans. (18) July 22.
Coal, Ash and Sand Handling Station, Illinois Central Railroad, Chicago, Ill. (18) July 29.
The North Bessemer Tunnel. F. E. House. (18) July 15; (15) July 7.
The New Treptow-Stralau Tunnel under the Spree. (19) May 20.
An American Railway in Colombia. Francis R. Hart. (7) June, 1899.
A Railway Pile-Driver. (4) June, 1899.
Preservation of Timber. Samuel M. Rowe, M. Am. Soc. C. E. (4) June, 1899; (13) July 20.
What is a Light Railway? Arthur Cadlick Pain. (11) June 9; (12) June 9.
Mountain Railways. Charles Assheton Whately Pownall. (11) June 9; (12) June 9.
Earth Slips. Robert Elliott Cooper. (11) June 23.
The Transport of Minerals. Charles De Neuville Forman. (11) June 23.
The Transportation of Minerals by Rail. J. D. Twinberrow. (11) June 9.
Through Transportation on Common Roads and Street Railways. (13) June 15.
Some Principles Governing the Proper Distribution of Locomotives in Economical Railway Operation. E. M. Herr. (25) June, 1899.
Economics of Freight Locomotive Operation. G. R. Henderson. (25) June, 1890.
The Function of a Railroad Testing Laboratory. Samuel Stockton Voorhees. (1) June, 1899; (15) August 4.
Building the Niagara Gorge Railway. George A. Ricker. (2) July, 1899.
The Problem of Elevated and Suburban Electric Railway Transportation. Frank J. Sprague. (17) July, 1899; (15 July 7.
Motor Power on Tramways. Thomas Parker. (11) June 9; (12) June. 9.
An Interesting Piece of Railway Location (India). (18) May 20.
Right of Way, Grades and Grade Crossings of Electric Railways. (13) May 18.
A Peculiar Grade Crossing, Switzerland. (13) June 15.
Change of Gauge on the Columbia and Western Railroad (Canadian Pacific Railway System). (13) July 20.
The Present Status of Electric Welding of Rails. (17) June, 1899.
Relaying Railways. Harry Footner. (11) June 23.
The Effects of Wear Upon Steel Rails. William George Kirkaldy. (63) Part II, 1899.
Comparison of Cost and Efficiency of Various Styles of Small Railway Culverts. Hunter McDonald, M. Am. Soc. C. E. (18) June 3.
Street Railway Track Construction. M. D. Burke. (1) April, 1899.
Track Construction of the Cincinnati Street Railway Company. (17) July, 1899.
Steel *vs.* Wood Tanks. Theodore W. Snow. (4) June, 1899.
Standard Steel Water Tanks of the Atchison, Topeka and Santa Fé. (15) July 21.
Railway Signaling. Alexander Ross. (11) June 9; (12) June 9.
Audible Station Signals (51) May 10.
Brierley's Patent Railway Signaling Apparatus. (21) July, 1899.
Controlled Manual Block Signals for Single Track. (15) June 2.
Electro-Pneumatic Signaling Plant, Bethnal Green. (11) June 16.
Block Signaling and Interlocking on the Chicago, St. Paul, Minneapolis and Omaha Railway. (13) May 25.
An Automatic Block System for Single-Track Electric Railways. (13) May 25.
Interlocking Plant at Pacific Junction, Chicago, Milwaukee and St. Paul Railway. (13) May 11; (18) May 13.
Bridge Signals at Rugby, London and Northwestern Railway. (21) May, 1899.
The Advantages or Disadvantages of Uniting the Lock and Block Signaling Appliances of Railways. Alexander Ross. (21) July, 1899.
The Wootten Boiler. (18) June 3.
Locomotive Boilers—Some Points. (15) June 9.
The Determination of the Friction Losses of Locomotives. (15) June 9.
Locating the Cut-Off and Other Events on Locomotive Indicator Cards. C. H. Robertson. (15) June 30.
Best Method of Applying Stay Bolts to Locomotive Boilers. (13) June 29.
Recent Practice in Purifying Feed Water for Locomotives. (13) June 29.
The Allen Valve for Locomotives. C. H. Quereau. (20) May 25; (13) June 15.
An Adjustable Valve Model. (18) July 15.
Piston Valves for Locomotives. F. M. Whyte. (25) June, 1899.
Locomotive Design—The Working Strength of Materials—Boilers. F. J. Cole. (25) June, 1899.
Notes on the Effect of Ports and Valve Gears on the Performance of High-Speed Locomotives. R. A. Smart. (25) June, 1899.
English and American Locomotive Building. Charles Rous-Marten. (9) July, 1899.
Geared Locomotives in the United States. (12) May 19.
Notes on Hot Main Pins. C. H. Quereau. (51) May 25.
Smokeless Firing on the Cincinnati, New Orleans and Texas Pacific. (15) July 21.
Compound Locomotives. Francis William Webb. (11) June 9; (12) June 9.
Player's Tandem Compound Locomotives. (25) June, 1899.
A Double-Ported Valve for Compound Locomotives. (40) June 30.
The Drifting of Two-Cylinder Compounds. (15) June 9.
Vauclain Compound *vs.* Simple Locomotive with Piston Valves. (25) August, 1899.

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The Atlantic Type (of Locomotive) and the Wide Firebox. J. Snowden Bell. (25) June, 1899.
Atlantic Type Locomotive, Pennsylvania Railroad. (15) July 7.
Atlantic Type Locomotives for the Erie. (15) July 21.
Ten-Wheel Passenger Locomotives for the Denver and Rio Grande Railroad. (15) July 28; (18) July 29.
Schenectady Ten-Wheel Passenger Engines for the New York Central. (15) August 4; (25) August, 1899.
Fast Passenger Locomotives of the Chicago and Northwestern Railway. (15) May 26: (25) June, 1899; (40) June 2; (18) June 3.
Recent Schenectady Locomotives. (40) May 19.
Heavy Passenger Locomotive for the Vandalia Line. (18) May 20.
The Burlington's Largest Passenger Locomotives. (40) May 12.
New Passenger Locomotives of the Pittsburg and Western Railway. (15) August 4.
Heavy Freight Locomotives for the Lehigh Valley Railroad. (13) July 27.
60-Ton Heisler Geared Locomotive; McClond River Railroad, Northern California. (13) July 27.
Freight Locomotives for the Lake Shore and Michigan Southern. (15) May 19.
New York Central Moguls in 1889 and 1899. (15) June 30.
Schenectady Consolidation Locomotives for the Delaware and Hudson. (15) July 28; (25) August, 1899.
Standard Consolidation Freight Locomotives, Pennsylvania Railroad, Classes H 5 and H 6. (25) June, 1899.
Express Passenger Oil-Burning Engine; Great Eastern Railway. (21) July, 1899.
Six-Wheels Coupled Goods Engine, Furness Railway. (12) July 7.
Six-Coupled Express Engines for Northeastern Railway. (12) July 21.
Express Passenger Locomotive for the London and Southwestern Railway. (11) June 16.
Stockton and Darlington Railway Engines. (21) May, 1899.
A Heavy Express Locomotive for the Lancashire and Yorkshire. J. Pearson Pattinson. (15) May 19.
Side Tank Passenger Engine, London and Northwestern Railway. (12) May 19.
American Locomotives for Foreign Railways. (13) June 15.
Tank Locomotive, Pretoria and Pietersburg Railway. (12) May 5.
The Automatic Freight-Car Coupler in the United States. H. G. Prout, M. Am. Soc. C. E. (10) June, 1899.
Laycock's Automatic Railway Coupling. (11) May 19.
Solid Wrought Forged Axle Box Frames *vs.* Riveted Patchwork Truck Frames. John A. Brill. (17) August, 1899.
Air Brakes on European Street Railways. (17) June, 1899.
Solid Brake Beams. R. H. Angier. (15) June 9.
Metal Plating for Passenger Coach. (18) July 22.
The Rolling Stock of the Great Central Railway. (11) July 14.
A Group of Instruction Cars. (40) May 12.
Meter-Gauge Composite and Third-Class Carriages, Eastern Bengal State Railway. (21) May, 1899.
Narrow Gauge Carriages, Eastern Bengal State Railway. (21) July, 1899.
Bogie Carriages, South-Eastern, and London, Chatham and Dover Railways. (12) June 2.
Bogie Mineral Wagons, 50 tons Capacity, Caledonian Railway. (21) July, 1899.
Oil Sprinkling Car, Boston and Maine Railway. (18) July 8.
Canda 100 000-Pound Capacity Cars. (15) June 9.
Norfolk and Western Railway 100 000-Pound Hopper Car. (18) June 10; (15) July 28; (25) June, 1899.
Steel Frame Hopper Cars. (40) June 2.
Insulated Meat Van; North-Eastern Railway. (21) June, 1899.
Car-Wheel Construction. Edward Grafstrom. (25) June, 1899.
The Manufacture of Car Wheels. G. R. Henderson. (17) June, 1899.
The Heating of Car Wheels and Its Relation to the Heat Test. (15) June 9.
Electric Lighting of Railway Trains in Great Britain. H. Scholey. (10) June, 1899.
Electric Intercommunication in Railway Trains. W. E. Langdon. (21) May, 1899.
Electric Locomotive Design. (11) May 26.
The Diatto System of Electric Traction. (12) May 26.
Electro-Magnetic Traction System. (17) June, 1899.
The Influence of Electricity Upon Railway Locomotion. B. H. Thwaite. (9) June, 1899.
The Electric Railways of Budapest. Alfred O. Dubsky. (10) June, 1899.
Electric Locomotives in Practice and Tractive Resistance in Tunnels. (17) June, 1899.
The Electric Railway System of Milwaukee and Eastern Wisconsin. (17) June, 1899.
Three-Phase Transmission for Railway Service Near Philadelphia. (17) June, 1899.
Speed Tests on the Cleveland-Lorain Railway. (17) June, 1899.
Edison Electric Railways of 1880 and 1882. Edwin W. Hammer. (27) June 10.
Mechanical Traction by Electricity. Granville C. Cunningham. (11) June 16.
The Hull Electrical Tramways. (26) July 7.
The Dudley and Stourbridge Electric Tramways. (26) July 28.
High Speed Electric Railway Between Dusseldorf and Krefeld. (17) August, 1899.
The Berlin Combined Trolley and Conduit Electric Railway. (13) August 3.
Compressed Air Motors in New York. (17) June, 1899.

Railroad—(Continued).

Labor-Saving Devices in Cincinnati Street Railway Repair Shops. (17) June, 1899.
Le Chemin de Fer du St. Gothard. Le Matérial de la Voie—Le Personnel de l'Exploitation. Jules Michel. (38) May 5.
La Nouvelle Gare Terminus de la Compagnie d'Orléans. (33) June 10.
Le Metropolitain de Paris. G. Leugny. (36) June 10.
Le Prolongement de la Ligne d'Orléans dans Paris. Emile Dieudonne. (36) June 25.
Les Travaux d'Agrandissement de la Gare de l'Est. G. Leugny. (36) July 10.
Note sur les Nouvelles Lignes de la Compagnie des Chemins de Fer de l'Ouest, dans Paris et dans la Banlieue. (38) June, 1899.
Application sur le Réseau du Nord des Serrures d'Enclenchement du System Bouré. M. A. Montier. (38) June, 1899.
Emploi des Courants Triphasés dans les Tramways Électriques. Louis Michel. (38) June, 1899.
Application de la Traction Méchanique au moyen d'Accumulateurs Électriques aux Lignes du Louvre à Vincennes de la Compagnie des Omnibus de Paris. (38) July, 1899.
Le Freinage des Trains de Marchandises. (36) June 10.
Locomotion sur Rails. Tramways "Système Serpollet et Système Walker." (36) July 10.
Applications de la Traction Électrique sur le Réseau de la Compagnie Paris-Lyon-Méditerranée. (33) July 22.
Note sur les Appareils de Vérouillage Électrique en usage à la Compagnie de Paris-Lyon-Méditerranée. M. Bouvier. (38) May 5.

Sanitary.

Sewers Laid under Aqueducts at Newton, Mass. Henry D. Woods, M. Am. Soc. C. E. (1) May, 1899.
The Sewerage of London, England. (14) June 3.
The Shone Sewerage System at Karachi. (14) May 27.
Tail Race of the Chicago Drainage Canal (13) July 6.
Twin Tide Gate for the Swanson Street Canal Outlet. Philadelphia, Pa. (13) July 20.
The Relation of Manufacturing Wastes to the Sewerage Problem. (13) May 25.
Sewage Disposal at Glasgow. James H. Fuertes, M. Am. Soc. C. E. (14) May 20.
Sewage Analysis and the Chemical Treatment of Sewage. Leonard P. Kinnicutt. (13) May 11.
Notes on the Berlin Sewage Farms. (14) June 10.
Sewage Sludge Disposal by Natural Agencies. G. Sims Woodhead. (11) June 16.
Sewage Disposal at Brockton, Mass. (14) July 8.
Septic Tank Sewerage Works at Barhead. (12) July 14.
A Sewage Distributing Tank and Automatic Dosing Apparatus for Marion, Ind. F. A. Barbour. (13) July 13.
English Experience with Garbage as Fuel for Electric Light Stations. (13) July 13.
Refuse Disposal in New York in 1898. (14) June 10.
The Purification of the Thames, England. (14) June 17.
State Drainage Work in the Red River Valley, Minnesota. (13) June 15.
The Toilet Conveniences in a Model Foundry. (13) July 27.
The Ventilation of Tunnels and Buildings. Francis Fox. (63) Part II, 1899.
The Disinfecting Station at Genoa. James H. Fuertes, M. Am. Soc. C. E. (14) July 22.
The Sanitary Condition and Requirements of Havana, Cuba. E. Sherman Gould, M. Am. Soc. C. E. (13) August 3.
Les Bains-Douches a Bon Marché à Rouen et à Paris, Paul Sarrey. (33) May 6.
Etat Actuel de l'Assainissement de Paris. (33) July 22.

Structural.

Chemical Tests of Cement. John F. Wixford and S. Bent Russell, M. Am. Soc. C. E. (1) June, 1899.
Experiments to Determine the Effect of Freezing on Concrete. Walter A. Rogers. (4) June, 1899.
The Utilization of Trass mixed with Hydraulic Cement in Germany. (13) July 27.
The Utilization of Volcanic Products; Basalt, Lava Stone and Trass. (12) May 26.
Some Features of the Structural Design of Buildings. William H. Burr, M. Am. Soc. C. E. (3) June, 1899.
Mechanical Influence in Architecture. G. W. Percy. (1) April, 1899.
Effect of Different Speeds and Radii of Curves upon the Inclination of Bicycle Tracks. John C. Quade. (4) April, 1899.
Wind Pressure on Buildings and other Structures. J. O. V. Irminger. (24) May 29.
The Dynamics of Framed Structures. H. Reisner. (49) VII-IX.
Groined Steel Roof Arches. (14) July 8.
A Novel Design for a Steel Bin for the Storage of Coal, Grain, Sand or Cement. A. S. Berquist. (13) July 27.
The Deadening of Building Walls, in order to Improve the Acoustics. R. Heger. (51) July 15.
New Features of Fireproofing. (14) July 1.
Fireproof Floor Construction in California. G. W. Percy. (14) June 17.
An Enclosed Spiral Fire Escape. (14) June 10.
Tests of Fireproof Construction by the British Fire Prevention Committee. (13) June 8.
Water Curtains for the Fire Protection of the Chicago Public Library. (13) June 8.
New House of the Danish Society of Engineers. (55) April 22.

Structural—(Continued).

The New Home of the Institution of Mechanical Engineers. (12) May 12.
The Design of the Public Schools of London. (14) July 22.
The Kansas City Convention Hall. (14) July 22.
The New Free Public Library, Jersey City, N. J. (14) May 20.
Glasgow International Exhibition, 1901. (12) May 5.
Remodeling the Grand Central Station, New York. (14) June 17.
Raising and Moving Large Buildings. (13) June 1.
Moving a Great Steel Arch. (20) July 20.
The Jersey City Trainshed. (14) August 5.
A Practical Ice Storage House. Kauser. (51) July 5.
Cantilever Foundations of an English Building. (14) August 5.
Steel Tank Grain Elevator on the Chicago and Western Indiana Railway. (18) July 15.
Plant of the Maryland Cement Company, Sparrows Point, Md. Frederick H. Lewis, M. Am. Soc. C. E. (14) July 15.
The New Building of the Weston Electrical Instrument Company. (20) July 13.
Two Noteworthy New York Stables. (14) July 15.
The Strawbridge Stable, Germantown, Philadelphia, Pa. (14) May 27.
Veterinary Stable, Cornell University, Ithaca, N. Y. (14) June 3.
The Equipment of Tall Office Buildings in New York City. (14) May 13.
The Power Plant of Columbia University. (14) May 13.
Some Engineering Features of a Philadelphia Residence. (14) June 17.
The Mechanical Plant of the Park Row Building, New York. (14) July 1.
The Distribution of Steam at Dartmouth College. (14) July 8.
A Gravity Steam Heating Apparatus in a Church. (14) July 8.
Heating with Steam at or Below Atmospheric Pressure. (14) July 29.
A Hospital Central Heating Plant. (14) July 22.
The Heating Plant of the University of Wisconsin. (14) May 13.
Engineering Features in the Alexian Brothers Hospital, Chicago. (13) July 20.
The New York Athletic Club's Swimming Bath. (14) May 20.
Plumbing in the New York Athletic Club House. (14) July 29.
Ventilation. Prof. R. H. Smith. (12) May 5.
Ventilation and Heating of a New York School. (14) May 27.
The Ventilation of a Newark Jail. (14) June 10.
Glue Testing. S. Rideal. (19) July 22.
Manganese in the Acid Martin Process. K. Toech. (50) June 15.
La Cheminée Monumentale de l'Usine la Bourdonnais au Champ-de-Mars, Exposition de 1900. Ch. Dantin. (33) May 6.
Évolutions des Constructions en Ciment Armé. (36) June 10.
La Nouvelle Gare de Tours. (33) June 24.
Théorie et Applications Nouvelles de Ciment Armé. Horel de la Noë. (43) First trimestre, 1899.
Constructions Diverses pour Determines la Poussée des Terres sur un Mur de Soutènement. M. Hisely. (43) First trimestre, 1899.
Palais des Fils, Tissus et Vêtements, au Champ-de-Mars, Exposition de 1900. L. Bourdain. (33) May 13.
Petites Maisons d'Habitation à Montrouge (Seine). E. Rivoalen. (35) May, 1899.
Le Métal Déployé a l'Exposition. (36) May 25.

Topographical.

Surveys in Tropical Forests. (14) June 3.
The Application of the Railroad Spiral to Race Tracks. William G. Raymond, M. Am. Soc. C. E (13) June 15.
Relief Map Construction. Frank G. White. (13) July 13.
Geodetic Work in Spitzbergen. Prof. J. H. Gore. (19) August 5.
Geodetic Observations in the United States. (19) July 15.
Geology of the Wachusett Dam and Wachusett Aqueduct Tunnel of the Metropolitan Water-Works in the Vicinity of Clinton, Mass. W. O. Crosby. (7) June, 1829.
The Geodetic Observatory at Middlesex Fells. George L. Hosmer. (7) June, 1899.

Water Supply.

The Colton Index to Hydraulics. (14) June 17.
The New Masonry Dam at Holyoke, Mass. (14) July 22.
The Construction of a Crib Dam for the Butte City Water Company. Eugene Carroll. (1) April, 1899.
The Indian River Dam. George W. Rafter, M. Am. Soc. C. E. Wallace Greenalch, Jun. Am. Soc. C. E., and Robert E. Horton. (13) May 18.
The Hollow Walls or Inspection Chamber of the Charonne Reservoir, Paris. (13) July 13.
A Lesson in Water-Works Management. (14) June 17.
Preventing Tastes in the Syracuse Water Supply. (14) May 20.
A Consideration of the Rate per Head per Day of Supply in Different Towns, and of the Causes which Increase or Diminish it in Particular Instances. James Watson. (11) June 16.
The Use of Filtered Flood Water. Mathew Wilson Hervey. (11) June 16.
Preventing Water Waste in Newark, N. J. (14) July 15.
Water Consumption and Waste in Great Britain. (14) July 15.
The Estimation of Suspended Matters in Water. (14) August 5

Water Supply—(Continued).

On Rapid Methods for the Estimation of the Weight of Suspended Matters in Turbid Waters. Charles L. Parmelee and Joseph W. Ellms. (7) June, 1899.
The Water-Works of Columbus, Ohio. (14) June 3.
The Washington Aqueduct, 1853-1898. William R. Hutton, M. Am. Soc. C. E. (14) July 29.
Compton Hill Water Tower, St. Louis. (14) August 5.
The New Water Supply System of Rockford, Ill. (13) July 13.
London Water Supply. (12) May 19.
Some Difficulties in Obtaining a Water Supply, Birmingham, Ala. (14) May 27.
A Large Artesian Well Plant at Camden, N. J. (13) May 11.
Water-Works for the City of Bergish-Gladbach. N. Ehlert. (48) June 17.
The New Water-Works in Dresden. Vocherot. (48) July 1.
Logarithmic Diagram for Flow of Water in Clean Cast-Iron Pipes Under Pressure. (13) July 6.
Second Test of a 14-Inch Riveted Steel Main at New Westminster, B. C. A. McL. Hawks, Jun. Am. Soc. C. E. (13) July 6.
The Manufacture and Inspection of Cast-Iron Pipes. Thomas H. Wiggin. (1) May, 1899.
Some Notable Australian Steel Pipe Lines. (13) June 29.
Experiments on Various Forms of Fire Hydrants. (13) May 11.
Notes on Suction Pipes. (14) May 13.
The Joints of Riveted Water Pipe. (14) June 10.
Thawing Frozen Water Pipes by Electricity. (14) May 20.
Cleaning Cast-Iron Water Pipe with Scrapers at St. John, N. B., and Boston, Mass. (13) July 20.
Cleaning Street Mains. (14) July 22.
Scraping Water Mains, Boston, Mass. (14) June 17.
The Results of the Meter System in Richmond, Va. (14) June 3.
The Present Status of Water Filtration. (14) May 27.
A New Coagulant for Mechanical Filters. (14) May 27.
The Operation of Water Filters in Germany. (14) June 10.
Test of a Mechanical Filter, East Providence, R. I. (14) July 1.
Test of Mechanical Filter at East Providence, R. I. Edmund B. Weston, M. Am. Soc. C. E. (13) July 13.
The Utilization of the Water Powers of Italy. Enrico Bignami. (9) August, 1899.
The Water Power Development at Chambly. Charles W. Haas. (27) June 3.
Development of the Montmorency Falls. E. M. Archibald. (27) June 17.
The Engineering Development of Porto Rico. H. M. Wilson, M. Am. Soc. C. E. (9) July, 1899.
Turbine Installation at Strensham Mills, England. (11) June 2.
The Hydraulic Features of the Chambly Water Power. (14) June 17.
Pumping Engines Driven by Water Power. Charles A. Hague, M. Am. Soc. C. E. (13) July 6.
The East Jersey Water Company's Hydraulic Plant at Little Falls. (14) July 8; (13) August 3.
Elements of Design Favorable to Speed Regulation in Plants Driven by Water Power. Allan V. Garratt. (13) July 27; (24) August 7.
Note sur les conditions de Resistance des Barrages de Réservoirs en Maçonnerie. M. Barbet. (43) First trimestre, 1899.

Waterways.

The Design of Breakwaters. John Watt Sandeman. (11) June 16.
Notes on the Effects of Waves on Breakwaters in Different Depths of Water. William Shield. (11) June 16.
The Delaware Breakwater. (14) July 8.
Westport Harbor, N. Z. Wave-Basin. Thomas Harold Rawson. (63) Part II, 1899.,
The Brussels International Congress on Navigation of 1898; the Bruges Ship-Canal; and New Works at Ostend and Antwerp. Leveson Francis Vernon-Harcourt. (63) Part II, 1899.
Canalization of the River Fulda from Cassel to Münden. Volkmann and Twiehaus. (49) VII-IX.
The Panama Canal, According to the latest Plans (53) June 30.
Floods in the Brisbane River; and a System of Predicting their Heights and Times. Cyrus John Richard Williams. (63) Part II, 1899.
The Teutschert-Czischeck System of Lifting Ships, without Locks or Sluices. L. Czischeck. (53) July 7.
The Operation of the Hotopp Locks on the Elbe-Trave Canal. H. Arnold. (48) May 27.
The Great Canal Lock at Henrichenburg. (19) July 1.
The Largest Dipper Dredge Ever Built. (13) July 13.
The New Bates Hydraulic Dredges for the Volga. (14) July 22.
Sand Pump Dredges. Anthony George Lyster. (11) June 16.
Failure of Dam at Minneapolis Due to Previous Weakening Through Ice Pressure. James W. Rickey. (13) May 11.
Failure of a Minneapolis Dam by Ice Pressure. (14) May 13.
The Ochoa Dam. (14) May 13.
Partial Failure of the Timber Crib Dam of the Montana Power Company near Butte, Montana. M. S. Parker, M. Am. Soc. C. E. (1) April, 1899.
Le Canal de Marseille au Rhone. (36) May 25.
Note sur Pont-Rivière D'Oudan. M. Mazoyer. (43) First trimestre, 1899.
Construction du Port d'Escale de Zeebrugge; Note sur les Chantiers de l'Enterprise de MM. Coiseau et Cousin. Ch. Piens. (30) June, 1899.

NEW BOOKS OF THE MONTH.

Unless otherwise specified, books in this list have been donated to the Library by the Publisher.

AN ELEMENTARY AND PRACTICAL TREATISE ON BRIDGE BUILDING.

By S. Whipple. Cloth, 9 x 6 ins., 352 pp., illus. New York, D. Van Nostrand Company. $4.00.

This is a reprint of the second edition of this work. It treats of essentially the same general plans, combinations and proportions for bridge work as were discussed in its predecessor.

DE PONTIBUS.

A Pocket-Book for Bridge Engineers. By J. A. L. Waddell, M. Am. Soc. C. E. Morocco, flap, $6\frac{3}{4}$ x $4\frac{1}{2}$ ins., 403 pp., illus. New York, John Wiley & Sons, 1899. $3.00. (Donated by the Author.)

The author has endeavored in this work to treat essentially of the fundamental principles of bridge-designing and their application, and has drawn almost entirely upon his own professional practice for examples to illustrate the text. Three chapters are devoted to designing, eight to various classes of bridges, trestles and viaducts and elevated railroads, five to specifications and one each to detailing, the compromise standard system of live loads for railway bridges and equivalents for the same, timber trestles, inspection of materials and workmanship, designing of piers, triangulation and office practice. There are ten plates of curves and diagrams, eighteen tables and an index of thirty-one pages.

TABLES OF MOMENTS OF INERTIA

And Squares of Radii of Gyration; Supplemented by Others on the Ultimate and Safe Strength of Wrought Iron Columns, Safe Strength of Timber Beams and Constants for Readily Obtaining the Shearing Stresses, Reactions and Bending Moments in Swing Bridges. By Frank C. Osborn, M. Am. Soc. C. E. Fourth Edition. Morocco, 6 x 4 ins., 175 pp. New York, Engineering News Publishing Company, 1894. $3.00. (Donated by the Author.)

The aim of the author has been to supply a need felt by engineers and draftsmen engaged in structural work in iron and steel. In this edition, tables for facilitating the computation of stresses in swing bridges have been added.

ELECTRICITY IN AMERICA.

L'Électricité en Amérique. Notes de Voyage sur le Développement des Applications de l'Électricité aux Etats-Unis et au Canada. Par Marcel Delmas. Paper, $9\frac{1}{2}$ x $6\frac{1}{4}$ ins., 81 pp., illus. Paris, Publications du Journal *Le Génie Civil.* (Donated by the Author.)

This pamphlet is a reprint from the author's contribution to *Le Génie Civil* of his observations on the development of the electric industries in North America. The subject is considered under the headings: Electric traction on metropolitan lines and tramways and on railways; electric lighting; workshops of construction; transmission of energy; electro-chemistry; accumulators; some applications; electrical instruction; conclusion.

TEMPERATURE COEFFICIENTS OF "CONDUCTIVITY" COPPER.

By Clark, Forde & Taylor. Roan, 8 x 5 ins., 15 pp. London, Waterlow & Sons, Limited. 1899.

This pamphlet is the result of a series of observations by the potentiometer method on six samples of copper wire at five or six different temperatures. The formula deduced is accompanied by a diagram and three tables to facilitate its use.

MECHANICS.

The Elementary Principles of Mechanics. By A. Jay Du Bois. Cloth, 9 x 6 ins., 3 Vols. New York, John Wiley & Sons, 1894–95. $11.00. (Donated by the Author and the Publishers.)

The author designs this as a text book for technical schools. Articles for advanced students are in small type. Articles containing applications of the calculus are enclosed in brackets. The matter in large type by itself constitutes an abridged course. The contents are: Volume 1, Kinematics; Volume 2, Statics; Volume 3, Kinetics. Each volume is furnished with an index.

HANDBOOK OF ENGINE AND BOILER TRIALS

And of the Indicator and Prony Brake; for Engineers and Technical Schools. By R. H. Thurston, Fourth and Revised Edition. Cloth, 9 x 6 ins., 604 pp., illus. New York, John Wiley & Sons, 1897. $5.00. (Donated by the Author.)

The author endeavors to present those methods of trial of heat-engines which have become standard; to exhibit the processes of their application; to describe the best form of apparatus used to date in conducting them, and in securing the data sought; to illustrate their use and their various capabilities; and to present examples of the reports made by distinguished engineers on work of this character.

TABLES FROM THURSTON'S STEAM-ENGINE MANUAL.

Handy Tables, from Thurston's Steam-Engine Manual, for Use in Laboratory Computations in Technical Schools; Numerical Trigonometrical and Thermodynamic Quantities, Steam and Horse-Power Tables, and Record-Forms for Laboratory Use. By R. H. Thurston. Cloth, 9 x 6 ins., 69 pp. New York, John Wiley & Sons, 1891. $1.50. (Donated by the Author.)

The contents are: Numerical Constants, Circles, Areas, etc.; Logarithms, Common and Natural; Mean Pressure Ratios; Terminal Pressures; Heat Transfer and Transformation; Comparison of Thermometers; Volumes of Water; Densities; Metric Steam Table; Metric Steam and Work Table; Steam Table, British Units; Stored Energy in Steam and Water; Formulas for Properties of Steam; Factors of Evaporation; Composition of Fuels; Horse-Power Constants; Real Ratios of Expansion; Logs and Forms for Blanks; Electrical Horse-Power; Water Computation Tables; Hirn's Analysis Blanks; Heat and Power Utilization, Non-condensing Engine.

LES MOTEURS LÉGERS

Applicables à l'Industrie aux Cycles et Automobiles, à la Navigation à l'Aéronautique, à l'Aviation, etc. By H. de Graffigny. Paper, 10 x 6½ ins., 335 pp., illus. Paris, E. Bernard et Cie. 10 francs.

This book on light motors has for its object the presentation of a *résumé* of all that has been proposed by inventors and accomplished by mechanics in order to show in limited space the value and importance of the several forms of motor mechanism and the various power generators used therewith. Steam, petroleum and electricity as motive powers are presented in turn. The chapter headings are: Les Générateurs à Vapeur; Les Nouveaux Moteurs à Vapeur à Pistons; Machines à Vapeur Rotatives et Turbo-Moteurs; Moteurs à Gaz, à Pétrole et à Benzine; Les Carburateurs; Les Moteurs à Pétrole Industriels; Les Moteurs d'Automobiles; Les Petits Moteurs à Essence de Pétrole pour Motocycles; Les Moteurs Rotatifs à Pétrole; Les Générateurs d'Electricité; Les Moteurs Electriques; Moteurs Divers. There is an appendix of eleven pages of various tables of capacities and an index of seven pages.

MOTIVE POWER OF HEAT.

Reflections on the Motive Power of Heat, from the Original French of N. L. S. Carnot; accompanied by an Account of Carnot's Theory, by Sir William Thomson (Lord Kelvin). Edited by R. H. Thurston.

Second, Revised, Edition. Cloth, 8 x 5 ins., 262 pp. New York, John Wiley & Sons, 1897. $2.00. (Donated by the Editor.)

The preface states that the present translation of Carnot's work is published only as a matter of limited but most intense scientific interest, as the work itself has been long out of date as a scientific authority. The purpose is to make it accessible to all who desire to study it in English, and preserve a permanent memorial to the founder of thermodynamics. The contents are: The Work of N. L. Sadi Carnot, by the Editor: The Life of N. L. Sadi Carnot, by H. Carnot: Reflections on the Motive Power of Heat and on Machines Fitted to Develop that Power, by N. L. Sadi Carnot: Account of Carnot's Theory, by Sir William Thomson (Lord Kelvin): Appendix—Extracts from Unpublished Writings of Carnot; Carnot's Foot-Notes, Note by the Editor.

ENERGY AND HEAT.

Supplement to Energy and Heat. The Thermometric Scale. Combustion—A Theory. By John Roger. Paper, 7 x 4¾ ins., 17 pp., one diagram. New York, Spon & Chamberlain. 25 cents.

This pamphlet comprises two studies: one suggests a thermometric scale which will by each degree indicate the same amount of heat in a body of uniform volume and the other presents a theoretical consideration of the phenomena of heat.

WASTE OF ENERGY BY FRICTION.

A Treatise on Friction and Lost Work in Machinery and Millwork. By Robert H. Thurston. Sixth Edition. Cloth, 9 x 6 ins., 380 pp., illus. New York, John Wiley & Sons, 1898. $3.00. (Donated by the Author.)

The work is the result of a series of investigations exhibiting concisely the principles involved in the transmission of power and the performance of work, and in the waste of power by friction. The last chapter, which treats of the real value of lubricants, contains a development of principles given in an earlier work but here worked out into a consistent algebraic theory; with illustrations of its practical application. The contents are: Theory of Machinery—Its Action and Its Efficiency; Nature Laws, and Theory of Friction; Lubricants; Lubrication—Apparatus; Chemical and Physical Tests of Oils; Experiments on Friction—Testing Machines; Lubricated Surfaces—Coefficients of Friction—Modifying Conditions; The Finance of Lost Work of Friction.

A TEXT BOOK ON ROADS AND PAVEMENTS.

By Fred. P. Spalding, M. Am. Soc. C. E. Cloth, 7¼ x 5 ins., 213 pp., illus. New York, John Wiley & Sons. $2.00.

The aim of the author of this book is to give a brief discussion, from an engineering standpoint, of the principles involved in highway work, and to outline the more important systems of construction, with a view to forming a text which may serve as a basis for a systematic study of the subject. The chapter headings are: General Considerations; Drainage of Streets and Roads; Location of Country Roads; Improvement of Country Roads; Broken-Stone Roads; Foundations for Pavements; Brick Pavements; Asphalt Pavements; Wood Pavements; Stone-Block Pavements; City Streets.

VITRIFIED BRICK FOR STREET PAVEMENTS.

A Description of its Manufacture and Some Notes on its Use. By Frederic A. Kummer, Jun. Am. Soc. C. E. Cloth, 8 x 5 in, 46 pp. Catskill, N. Y., 1899. (Donated by the Author.)

In presenting this work, the author desires to point out concisely and clearly the advantages of brick as a street paving material. The contents are: Composition of Paving Brick; Paving Materials; The Selection of Paving Brick; Laying the Brick—The Foundations, The Joints; Specifications for Laying Brick.

THE ELEMENTS OF RAILROADING.

By Charles Paine. Cloth, 7¾ x 5 ins., 154 pp. New York. The Railroad Gazette. $1.00. (Donated by the Author.)

This is a series of short essays reprinted from *The Railroad Gazette*, and dealing with various branches of the subject from the organization of the engineering party for the survey of a new road to the operation of the finished line and the selection and discipline of employees. Chapter headings are: Surveying and Construction; Real Estate and Records; Drainage; Main Track; Trackmen and Sidings; Stations; Shops and Engine Houses; Telegraph Lines and Fences; Locomotives; Cars; The Movement of Freight; The Movement of Passengers; Employees.

THE RAILWAY TRANSITION SPIRAL.

By Arthur N. Talbot, M. Am. Soc. C. E. Paper, 6½ x 4 ins., 64 pp., illus. Published by the Author. University of Illinois. Champaign or Urbana, Ill.

This article was originally published in *The Technograph* for 1890-91. The adoption of the method described on many railroads and the demands for the original issue, now out of print, led to this republication with modifications and extensions. It is aimed to so treat the principles involved that a knowledge of the demonstrations of the formulas is not essential to their comprehension, and the methods used in laying out the spiral are designed to be mastered by instrument men without difficulty. There are 11 pages of tables.

RAILWAY TRACK AND TRACK WORK.

By E. E. Russell Tratman, Assoc. M. Am. Soc. C. E. Cloth, 9 x 6 ins., 400 pp., illus. New York, The Engineering News Publishing Company. $3.00. (Donated by the Author.)

This book is intended to treat the subjects of Track and Track Work, into which two parts it is divided, in a comprehensive manner. It includes not only systems everywhere applicable and the general principles underlying track work, but also numerous details of material, appliances and work and varying methods of practice, intended to be fully representative of standards on different roads in all parts of the country. The various subjects are treated, not only in a descriptive, but also in a critical, manner. There are 25 chapters, over 200 illustrations, appendixes giving tables of standard track construction on 50 railways and of train speeds and distances run, and an index of four pages.

EARTHWORK MEASUREMENT.

Diagrams for the Graphic Calculation of Earthwork Quantities. By Alfred Henry Roberts, C. E. Leather board case, flaps, 14½ x 9 ins., 5 pp., 10 cardboard diagram plates. London, Robt. J. Cook & Hammond. (Donated by the Author.)

These diagrams have been prepared with the object of enabling engineers, contractors and others to arrive at fairly accurate estimates of earthwork quantities: (1) When no cross-sections are available, or, (2) When it is desirable to obtain quantities rapidly without the slow process of taking out the area of each cross-section and of multiplying out each item. Of the ten diagram plates three are applicable to level cuttings at depths varying from 0 to 60 feet and seven to sidelong ground for widths varying from 15 to 30 feet. The diagrams are intended to be suitable not only for railway cuttings, but for any straightforward excavation, such as canals, stream diversions, pipe trenches, railway widenings, docks, etc.

THE SEWERAGE ENGINEER'S NOTE BOOK.

Being Standard Notes on Sewer Formulas and Sewerage Calculations. By Albert Wollheim, A. M. Inst. C. E. Boards, 6½ x 4¼ inches. 140 pp., illus. London, The St. Bride's Press, Limited. 2 shillings and 6 pence.

To the notes and data originally collected and compiled by the author for his own use other tables and 27 diagrams have been added. The publication is intended as a handy book for the sewerage engineer, the assistant and student. It is in five parts, each followed by blank pages for memoranda by the reader, and most of the tables are grouped in the appendix, which occupies 75 pages.

EXAMINATIONS IN SANITARY SCIENCE.

By Ernest R. Matthews. Boards, 6½ x 4¼ inches. 44 pp., illus. London, The St. Bride's Press, Limited. 6 pence.

These are questions set at the examinations of the Sanitary Institute, the Surveyors' Institution, the Incorporated Institution of Municipal and County Engineers, etc., with numerous conditions and suggestive answers, and are intended to be of service chiefly to students who are preparing for such examinations.

HOUSE DRAINAGE:

Its Inspection and Testing. By Richard J. Jenkins. Boards, 6½ x 4¼ ins., 43 pp., illus. London, The St. Bride's Press, Limited. 1 shilling.

In this book the author has endeavored to point out in a concise form the most important principles of the construction of drains, and to describe the various methods of testing. Appendixes treat of the size, inclination, ventilation and planning of house drains.

FLOW OF SEWAGE.

By John P. Dalton. Card board, 6½ x 4¼ ins., 4 pp. London, The St. Bride's Press, Limited. 6 pence.

On this card there is reprinted, from *The Surveyor and Municipal and County Engineer*, formulas for circular and egg-shaped sewers, together with tabular numbers and their logarithms for sewers of both forms running from one-quarter full to full, and also the fifth powers and logarithms of d or r.

CIRCULAR SEWER DISCHARGES.

Table of Discharge from Circular Sewers when Running Full, in Cubic Feet per Minute. By A Municipal Engineer. Boards, 4¼ x 3¼ inches. 39 pp. London, The St. Bride's Press, Limited.

This little pocket-book has been compiled to aid a municipal engineer in preparing sewerage schemes and in estimating the efficiency of existing sewers. Besides giving the size and inclination of sewer necessary to take a given amount of sewage per minute, it indicates the least fall which may safely be given to a sewer of a certain size to induce a self-cleansing velocity of flow, by omitting all figures below that limit.

SEWAGE DISPOSAL.

Description of Various Sewage Disposal Works. A Report prepared by R. O. Wynne Roberts, Am. Inst. C. E. Boards, 6½ x 4¼ inches. 105 pp., illus. London, The St. Bride's Press, Limited. 1 shilling.

This is the substance of a report made to the Oswestry Town Council upon representative sewage disposal works. The systems described embrace discharge into the sea or a tidal river, land irrigation with crude sewage and with crude sewage after partial separation of the sludge, chemical clarification and land irrigation, chemical clarification and artificial filters, electrical treatment, biological treatment and the septic tank system. Tables of cost, efficiency and expense of operations are appended in some cases.

SEWAGE DISPOSAL.

A Discussion of the Prevailing Theories and Practices Relating to Sewage Disposal. By Wynkoop Kiersted, M. Am. Soc. C. E. Cloth, 7¼ x 5 ins., 182 pp. New York, John Wiley & Sons. $1.25. (Donated by the Author.)

Nature, says the author in his preface, gives conclusive evidence of the existence of a natural process of purification of polluted waters, of such a character that it can be utilized in the purification of sewage. The author's aim in this book is to set forth this natural process of purification, which with the present methods of sewerage and drainage must become the foundation of any successful method of sewage disposal, and to treat the various considerations embraced in questions of sewage disposal in a simple and practical manner. There is an index of eight pages.

CHIMNEY DESIGN AND THEORY.

A Book for Engineers and Architects. By William Wallace Christie. Cloth, 9 x 6 ins., 164 pp., illus. New York, D. Van Nostrand Company. $3.00.

This work is intended to present the latest practice in regard to the theory and designing of chimneys as built in the United States. The contents are: Introduction and History; Theory of Chimney Draft; Chimney Formulas; Chimney Tables, Wind-Pressure, Air-Space in Grates; Foundation Materials, Brick Chimney Materials; Steel Chimneys—Theory pertaining to same, and Examples from Existing Structures; Brick Chimneys—Theory pertaining to same, and Examples from Existing Structures; Chimney Performances—Special Types; Straightening Chimneys—Flues; House Chimneys; Lightning Protection; General Information.

THE HOUSING OF THE LABORING CLASSES

And Back-to-Back Houses. By H. Percy Boulnois, M. Inst. C. E., City Engineer of Liverpool. Boards, 6½ x 4¼ inches, 46 pp., illus. London, The St. Bride's Press, Limited. 1 shilling.

This book endeavors to show what has been done to meet the want of suitable housing for the very poor or laboring classes of large cities, and to give some suggestions as to the description of house which is required. The illustrations include sketch plans of dwellings in English and continental cities as well as suggested arrangements for such premises.

PARTY STRUCTURES.

London Building Act, 1894. Part VIII. By Sydney Perks. Boards, 6½ x 4¼ inches. 63 pp., illus. London, The St. Bride's Press, Limited. 1 shilling.

This work, which is intended to be of use to architects and surveyors, is confined to a consideration of the 15 out of 218 sections of the London Building Act which have reference to party walls.

THE TOPOGRAPHER,

His Instruments and Methods; Designed for the Use of Students, Amateur Topographers, Surveyors, Engineers, and All Persons Interested in the Location and Construction of Works based upon Topography. By Lewis M. Haupt, M. Am. Soc. C. E. Cloth, 9 x 6 ins., 247 pp., illus. Philadelphia, Henry Carey Baird & Co., 1883. $4.00. (Donated by the Author.)

It has been the author's aim to prepare a compact manual of practical value to the topographer, freed from all information so readily obtainable from ordinary sources, and containing, in condensed form, a complete course for the guidance of the amateur. The contents are: What and How to Observe; The Instrumental Outfit; Scales of Maps; Forms of Record; Graphical Representations; Reconnaissance, Linear; Instruments and Methods; On Determining a True Meridian; Leveling; Surveying and Filling In; Hydrography; Submerged Topography; Computations; Modeling; Field Sketching; Applications.

ENGINEERS' SURVEYING INSTRUMENTS.

Their Construction, Adjustment and Use. By Ira O. Baker, M. Am. Soc. C. E. Second Edition, Revised and Enlarged. New York, John Wiley & Sons, 1897. $3.00. (Donated by the Author and the Publishers.)

The object of this volume is to acquaint the student with the construction, adjustment and use of surveying instruments. It is not intended as a treatise on surveying. The contents are: Chain and Tape; Tripod, Leveling Screws, and Plumb-Bob; Magnetic Compass; Solar Compass; Verniers; Optical Parts of the Telescope; The Transit; Solar Transit; Plane Table; Telemeters; Spirit Levels; Barometers.

HYDRAULIC ENGINEERING.

A Practical Treatise on Hydraulic and Water-Supply Engineering Relating to the Hydrology, Hydrodynamics and Practical Construction of Water-Works in North America. By J. T. Fanning, M. Am. Soc. C. E. Fourteenth Edition, revised, enlarged, and new tables and illustrations added. Cloth, 9 x 6 ins., 644 pp., illus. New York, D. Van Nostrand Company. $5.00. (Donated by the Author).

This book aims to supply water commissioners with a general review of the best methods practiced in supplying cities and towns with water, and with facts and suggestions that will enable them to compare intelligently the merits and objectionable features of the different potable water sources within their reach; to present to junior and assistant hydraulic engineers a condensed summary of those elementary theoretical principles and the involved formulas adapted to modern practice, which they will frequently have to apply; to construct and gather, for the convenience of older practitioners numerous tables and statistics that will facilitate their calculations and to present to civil engineers generally a concise reference manual of the water-supply branch of their profession. The work is intended more especially for those who have already had a task assigned them, and it is designedly practical in text, illustration and arrangement. In the later editions some revised and some new tables and formulas have been added. There are twenty-six chapters, 150 tables, 245 illustrations, 23 pages of appendixes of engineering data and 11 pages of index.

RETAINING WALLS FOR EARTH.

Including the Theory of Earth-Pressure as Developed from the Ellipse of Stress, with a short Treatise on Foundations, Illustrated with Examples from Practice. By Malverd A. Howe, M. Am. Soc. C. E. Third Edition, Revised and Enlarged. Cloth, 7½ x 5 ins., 132 pp., illus. New York, John Wiley & Sons. $1.25. (Donated by the Author.)

In this work an attempt has been made to present the theory in a shape easily followed by those who have only a knowledge of algebra, geometry and trigonometry. In this edition a number of examples illustrating profiles of retaining walls and types of foundations selected from existing structures have been included. The treatise on foundations is intended to be sufficiently complete for the use of technical schools, if supplemented by lectures or by reference to descriptions of existing structures. The book is divided into: Theory of Earth-Pressure; Formulas for Earth-Pressure; Stability of Trapezoidal Walls; Formulas for Trapezoidal and Triangular Walls; Foundations for Walls Retaining Earth; Examples; Foundations; Foundations Under Water and Deep Foundations. There is a list of references and 26 pages of tables.

MATERIALS OF CONSTRUCTION.

A Text-Book of the Materials of Construction, For Use In Technical and Engineering Schools. By Robert H. Thurston. Fifth Edition. Cloth, 9 x 6 ins., 765 pp. New York, John Wiley & Sons, 1895. $5.00. (Donated by the Author.)

This volume is an abridgment of the larger work of the author, "Materials of Engineering." It is intended as a text-book for use in private reading. The essential parts of the earlier treatise have been retained. The contents are: Qualities of the Metals; Iron and Its Ores; Reduction of Ores; Production of Cast Iron; Manufacture of Wrought Iron; Manufacture of Steel; Chemical and Physical Properties of Iron and Steel; The Non-Ferrous Metals; The Bronzes; The Brasses; Kalchoids and Miscellaneous Alloys; Strength, Elasticity and Ductility of Metals; Stones and Cements; Timber; Strength and Uses of Timber; Miscellaneous Materials. There is an index of 67 pages.

TESTING OF PORTLAND CEMENT

For Smaller Municipal Works. By H. H. Humphreys, A. M. Inst. C. E. Boards, 6½ x 4¼ ins., 16 pp. London, The St. Bride's Press, Limited. 1 shilling.

This paper has been republished from *The Surveyor and Municipal and County Engineer* with the hope that it may afford practical help to young engineers in charge of construction work. A diagram of average tests of two makes of English Portland cements is included in a folding plate.

A PRIMER OF THE CALCULUS.

By E. Sherman Gould, M. Am. Soc. C. E. Second Edition, Revised and Enlarged. Boards, 6 x 4 ins., 122 pp., 24 folding plates. New York, D. Van Nostrand Company. 50 cents.

This work, which is No. 112 in the Van Nostrand Science Series, is a development of the infinitesimal calculus as far as the first differentials of algebraic functions of one independent variable and their corresponding integrals. It has been endeavored to teach a few elementary rules and then put them into immediate use. In the second edition, a number of practical applications of elementary calculus have been included in an additional chapter.

TABLES AND DATA.

For the Use of Students in Laboratories; also for Class and Home Work. By W. W. F. Pullen, Assoc. M. Inst. C. E. Cloth, 9 x 6 ins., 87 pp., illus. Manchester, Eng., The Scientific Publishing Co. 4 shillings.

This collection of tables and data for the mechanical engineer was made primarily to supply a want experienced in engineering laboratory work, and is intended to contain only that information which would be likely to be of use to the engineering student. The general steam table is carried by pound increments to 300 pounds per square inch. The tables and diagrams are supplemented by a series of mathematical notes inserted "to aid in filling the gap which fickle memory is so prone to provide."

METRICAL TABLES FOR ENGINEERS,

Surveyors, Chemists, Merchants, etc. By Frederic Elie Gay. Boards, $6\frac{1}{2}$ x $4\frac{1}{4}$ ins., 56 pp. London, The St. Bride's Press, Limited. 2 shillings.

This compilation was prepared to supply the want of a concise and portable series of tables which would transpose into English equivalents the values met with in French and Continental technical works. There are 192 tables included, most of them on the reciprocal basis, and all founded on the standards of the Weights and Measures Act of 1878. The tabular numbers have been carried in some instances to twelve decimal places in order that, when necessary, accurate results may be arrived at.

A FRENCH-ENGLISH TECHNICAL DICTIONARY.

Dictionnaire Technique Français-Anglais des Outils et Utensiles Employés dans les Métiers Manuels, la Petite Industrie, le Ménage, etc., suivi d'un Index Anglais-Français. By A. S. Lovendal. Paper, $7\frac{1}{4}$ x $4\frac{1}{2}$ ins., 158 pp. Paris, Boyveau et Chevillet. 3 francs.

This dictionary gives the common names in French and English of a number of tools used in the manual industries and is intended to include words not found in technical dictionaries heretofore published. The definitions are classified under thirty-eight heads referring to the kind of tools or to the industry in which they are used, and an English index is added.

COMMERCIAL MANAGEMENT

Of Engineering Works. By Francis G. Burton. Cloth, 9 x 6 ins., 310 pp. Manchester, Eng., The Scientific Publishing Company. 12 shillings 6 pence.

This treatise is designed to suggest to the youthful engineer the difficulties he will encounter whenever commercial duties are thrust upon him, and to outline measures which may assist him, without laying down any absolute rules to be adhered to under all circumstances. The organization of works and duties of officers, purchase and inspection of material, cost accounts, sales department, estimates, and the works-manager and dealings with workpeople are all taken up in turn, and the endeavor has been made in the suggestions offered to allow sufficient elasticity for most establishments of moderate size and for ordinary engineering undertakings. There is an index of 12 pages.

RENSSELAER POLYTECHNIC INSTITUTE.

Biographical Record of the Officers and Graduates of the Rensselaer Polytechnic Institute, 1824–1886. Edited by Henry B. Nason, Secretary of the Association of Graduates, with an introduction by Benjamin H. Hall. Cloth, 9 x 6 ins., 614 pp., illus. Troy, N. Y., William H. Young, 1887. $2.00. (Donated by the Institute.)

This record was authorized by the Association of Graduates in 1873 and the biographies cover the historical facts available referring to the classes of 62 years. The book contains a historical sketch of the Institute, a biography of the Hon. Stephen Van Rensselaer, lists and records of officers, trustees, professors and instructors, and records of graduates. In the biographical sketches an effort has been made to state facts concisely, without comment. A catalogue of non-graduates follows the biographical records, showing that the Institute has been useful to many who did not complete the course. There is a general index of ten pages and a separate index to the catalogue of non-graduates.

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ADDITIONS.

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Supt., Track, Bridges and Buildings, St. Louis and San Francisco R. R. Co., Springfield, Mo		May 3, 1899
BOND, FREDERICK WINN		
1806A California Ave., St. Louis, Mo		Dec. 7, 1898
BROWN, LINUS WEED		
Chf. Eng., National Contracting Co., 713 Hennen Bldg., New Orleans, La		June 7, 1899
BROWN, RALPH HENRY		
Chf. Eng., Boston Bridge Works, 9 Tenney St., Cambridge, Mass		June 7, 1899
CHURCHILL, JOHN CHARLES, Jr.		
U. S. Asst. Engr., Oswego, Oswego Co., N. Y		Feb. 1, 1899
COFFIN, THOMAS AMORY		
11 Taylor St., West New Brighton, Staten Island, N. Y	Jun.	Oct. 2, 1894
	M.	Feb. 1, 1899
FRANK, GEORGE WILLIAM		
Care of J. G. White & Co., 29 Broadway, New York City		Feb. 1, 1899
GILLETTE, LEWIS SINGER		
Prest., Gillette-Herzog Mfg. Co., Minneapolis, Minn		June 7, 1899
GRIFFIN, EUGENE		
44 Broad St., New York City		June 7, 1899
HARTRICK, EDWARD MACAULAY		
U. S. Asst. Eng., Galveston, Texas		Feb. 1, 1899
HILL, GEORGE	Assoc. M.	Oct. 4, 1893
Architect, 150 Fifth Ave., New York City	M.	Feb. 1, 1899
HIMES, ALBERT JAMES		
Asst. Eng., U. S. Board of Engrs. on Deep Waterways, 34 West Congress St., Detroit, Mich	Assoc. M.	Nov. 6, 1895
	M.	June 7, 1899
HUTCHINGS, WILLIAM EVELYN		
Asst. Eng., Chf. Engr.'s Office, Louisville and Nashville R. R., 1578 Barrett Ave., Louisville, Ky		June 7, 1899
MYERS, EDMUND TROWBRIDGE DANA, Jr.	Jun.	May 2, 1888
1103 Grove Ave., Richmond, Va	M.	June 7, 1899
PARKER, HAROLD		
Clinton, Mass		June 7, 1899
PERRILLIAT, ARSÈNE		
Asst. State Engr., Member, State Board of Engineers, New Orleans, La	Assoc. M.	June 7, 1893
	M.	April 5, 1899

		Date of Membership.
Ross, Douglas William		
State Engineer, Boise, Idaho		Mar. 1, 1899
Rust, Charles Henry		
City Engineer, Toronto, Ont., Canada		April 5, 1899
Stanton, Frank McMillan		
Atlantic Mine, Houghton Co., Mich		Feb. 1, 1899
Stuart, Francis Lee		
Asst. Eng., Nicaragua Canal Comm., 1515 Thirty-first St., Washington, D. C		May 3, 1899
Thompson, Ellis Dunn		
Prin. Asst. Eng., U. S. Engr's. Office, 815 Witherspoon Bldg., Philadelphia, Pa		Feb. 1, 1899

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Care of The American Trading Co., 99 Concession, Kobe, Japan	Jun.	Oct. 1, 1895
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Ames, George Marshall		
City Engr., City Hall, Grand Rapids, Mich		June 7, 1899
Baum, George		
Supt. of Gas Appliances, Consolidated Gas Co. of New York; Res., 27 Orchard St., Bloomfield, N. J	Jun.	Jan. 4, 1892
	Assoc. M.	April 5, 1899
Beugler, Edwin James		
Second Asst. Eng. of Constr., N. Y., N. H. & H. R. R., 162 Norman St., Bridgeport, Conn	Jun.	June 19, 1891
	Assoc. M.	Feb. 1, 1899
Boller, Alfred Pancoast, Jr.		
Care of H. R. Worthington, Honolulu, H. I	Jun.	Oct. 2, 1894
	Assoc. M.	May 3, 1899
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406 Postal Telegraph Bldg., Kansas City, Mo		Feb. 1, 1899
Cleverdon, Henry Lawrence		
Winona, Greenland Co., Mich		Mar. 1, 1899
Condron, Theodore Lincoln		
Res. Eng., Pittsburg Testing Laboratory, Ltd., 1750 Monadnock Block, Chicago, Ill		Feb. 1, 1899
Eastwood, John Thompson		
Asst. Eng. in Charge of Edison Conduit Construction, Edison Electric Co., 317 Baronne St., New Orleans, La	Jun.	Mar. 6, 1894
	Assoc. M.	Feb. 1, 1899
Ford, Robert Henry Persse		
Central Vermont R. R., St. Albans, Vt	Jun.	Mar. 2, 1897
	Assoc. M.	June 7, 1899
Foster, Thomas J.		
522 West 123d St., New York City		Feb. 1, 1899
Frink, Fred Goodrich		
Prof. of Civ. Eng., Univ. of Idaho, Moscow, Idaho		May 3, 1899

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287 Fourth Ave., New York City		June 7, 1899
KENNEY, EDWARD FULBISTER	Jun.	Nov. 3, 1896
Asst. Eng., Pennsylvania R. R., Moore, Pa.	Assoc. M.	May 3, 1899
MASON, GEORGE COTNER		
Instructor in Civ. Eng., New York Univ.,	Jun.	Mar. 6, 1894
University Heights, New York City	Assoc. M.	May 3, 1899
PRESTON, HENRY WRAY		
Union Bridge Co., Athens, Pa		May 3, 1899
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Asst. Eng., New East River Bridge, 84	Jun.	Dec. 1, 1896
Broadway, Brooklyn, N. Y.	Assoc. M.	June 7, 1899
PUTNAM, GEORGE ROCKWELL		
U. S. Coast and Geodetic Survey, Washington, D. C.		Feb. 1, 1899
STICKNEY, GEORGE FETTER	Jun.	May 1, 1894
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	Date of Membership.
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217 Carlton Ave., Brooklyn, N. Y.	Jan. 31, 1899

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Lea, Samuel Hill..... .....Tampico, Ill.
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Lynch, Michael Lehane.....Room 1124, Guaranty Loan Bldg., Minneapolis, Minn.
McGrath, Wallace .....National Soldiers' Home, Va.
MacAllister, Dickinson.....Pres. and Gen. Mgr., The Met. West Side Elevated Ry. Co., 169 Jackson St., Room 1001, Chicago, Ill.
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INSUFFICIENT PROVISION FOR COUNTER-STRESSES IN RAILROAD BRIDGES.

By HENRY S. PRICHARD, M. Am. Soc. C. E.

TO BE PRESENTED SEPTEMBER 20TH, 1899.

Many specifications for railroad bridges do not make sufficient provision for counterstresses. The term counterstresses must be understood, in what follows, to apply, not only to the stresses in the counters, but to all stresses from the action of the combined dead and live loads (including the effect of shock, sudden application, etc.), which are of a reverse character to the stresses from dead load alone.

In a good modern specification the unit stresses allowed are so low that, except for the counterstresses, a bridge designed and built in accordance therewith could reasonably be expected to be able to carry without serious injury a live load at least twice as great as the live load specified.

In view of the great increase which has been and is now taking place in the weights of locomotives and trains, not to mention other pertinent considerations, the wisdom of requiring this seeming surplus of strength will hardly be questioned, yet, as regards counterstresses, many otherwise good specifications make no such provision for increased loads.

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This defect is probably due in most cases to the failure of the authors of the specifications to realize that a provision for possible increase in loading, which is adequate for all other stresses, may be entirely inadequate for counterstresses.

To assist in the consideration of this subject, the relations between loads and stresses are stated as follows:

Calling L the live-load stress, D the dead-load stress, A the sectional area, and S the stress per square inch, then for counterstresses $S = \frac{L - D}{A}$; for stresses other than counterstresses $S = \frac{L + D}{A}$.

From these equations the following propositions are evident:

For stresses other than counterstresses, if the live load is doubled, the stress per square inch cannot be more than doubled, so that if for the original live load the stress per square inch was less than half the safe stress, the stress per square inch would still be within safe limits if the load was doubled.

For counterstresses any increase in the live load must make a greater proportional increase in the stress per square inch. If the live-load stress is but slightly larger than the dead-load stress, then even a small increase in the live load may make a very large increase in the intensity of the counterstresses, while if the live load is slightly smaller than the dead load, a small increase in the live load may require counters where none were required under the original live load.

If the counterstresses in a bridge are obtained by simply subtracting the dead-load stresses from the reversed stresses for the specified live load, and the members subject thereto are proportioned by the specified stress per square inch, which is the way prescribed in many specifications, and a common practice among bridge designers, then such members will have a less factor of safety as regards increase in the live load than the other members of the bridge, and the trusses may in some panels have no proper provision whatever for any increase in the live load.

The absence of a regular provision for counterstresses does not necessarily result in immediate failure if the bridge is subjected to a load sufficient to produce counterstresses, as the stiffness of the chords and floor system, and the fact that the load is generally but a few moments in the critical position which would produce the counterstresses may save a bridge from seemingly impending failure.

A case in point, brought to the author's attention about a year ago, is that of a 300-ft. span, shown in Fig. 1. This span, for an unknown period, had been carrying loads which, for want of counters, would temporarily buckle the main diagonals "*A*," whenever a loaded train crossed the bridge.

As this bridge is of a common type, and has no special features or unusual stiffness in the chords and floor system tending to supply the place of counters, it is fair to suppose that most bridges would possess some capacity for sustaining loads if some of the counters necessary to a proper design were omitted. It is to be feared, however, that loads so carried produce dangerous strains in the chords and floor system which may in time cause failure. Ability to carry loads when there is not proper provision for the counterstresses, like the ability to resist stress after the elastic limit has been exceeded, is a very desirable property in a bridge, and may, in an emergency, save it from disastrous failure, but it should not be relied on for sustaining the traffic, either present or prospective.

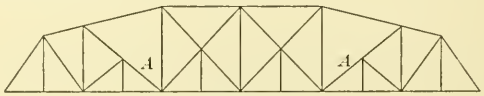

FIG. 1.

As a bridge designed in accordance with recognized good practice would be considered fairly safe as regards all stresses other than counterstresses, under increasing traffic up to a point where the live load was about double that originally specified, to be consistent, proper provision should be made for the counterstresses produced by a live load of at least double that originally specified. If the stresses are obtained from double the live load, the stress per square inch in members not subject to counterstresses will vary from something less than twice the specified stress per square inch in some members to nearly or quite twice the specified stress in others. Hence, it would seem to be consistent, after obtaining the counterstresses from double the specified live load, to allow nearly double the specified stress in proportioning.

All the previous references to live-load stresses are intended to be references to the live stresses, with the effect of shock, sudden application, etc., included. The author's present practice in determining what members of a bridge are liable to counterstresses, and in proportion-

ing them, is to add to the nominal live-load stresses (that is, the live-load stresses with the effect of shock, sudden application, etc., neglected), 125%, to combine the stresses thus increased with the dead-load stresses, and to use double the specified stress per square inch in determining the sectional area required.

Fortunately, even when the specifications require no provision for counterstresses beyond what would be produced by the specified live load, most bridge designers will make some provision, which, however, will differ greatly according to individual judgment. If the designers are contractors competing for work on a lump-sum basis, under specifications deficient as regards counterstresses, any provision they may make toward remedying such deficiency is a self-imposed handicap.

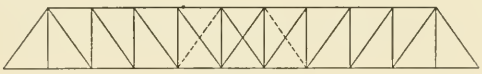

FIG. 2.

For bridges with parallel chords resting on two supports, as in Fig. 2, this handicap generally is not great, because for most cases only a few members will have counterstresses, even when the live load is doubled, and these stresses are mostly quite small; but for bridges with inclined chords, which for long and moderately long spans are now very frequently used, as in Figs. 1 and 3, the case is quite different, because the inclination of the chords decreases the dead-

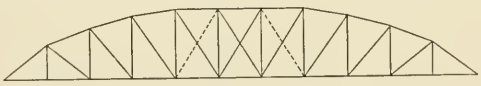

FIG. 3.

load stresses and increases the reverse live-load stresses in the web members, thus increasing the counterstresses. It is frequently customary to start the inclination of the chord at a sufficient distance from the center of the truss to avoid the use of counters in the portions of the truss having an inclined chord, and to make the inclination as great as the avoidance of counters and the maintenance of the required headroom will admit. As there is generally considerable economy in making the inclinations of the chords as steep as possible, there is a decided advantage, from the commercial point of view of

the aforesaid competing contractors, in making no provision for counterstresses in excess of the specifications.

The chief danger to be apprehended from specifications which do not make adequate provision for counterstresses lies, not in their use by those who from self-interest wilfully omit unspecified provisions which they believe to be essential to good design, but in their use by those who either fail to realize that the specifications are inadequate, or underestimate the extent of their inadequacy.

The same causes which lead engineers to write specifications which do not make sufficient provision for counterstresses may lead them to design bridges which are defective in this regard.

It is believed that what has been presented shows that provision should be made for counterstresses from some increase in the live load, perhaps more, perhaps less, than the increase advocated, and that whatever provision is decided on should in all cases be definitely and clearly stated in the specifications.

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COMPARISON OF WEIGHTS OF A THREE-HINGED AND A TWO-HINGED SPANDREL-BRACED PARABOLIC ARCH.

By C. W. HUDSON, Assoc. M. Am. Soc. C. E.

TO BE PRESENTED SEPTEMBER 20TH, 1899.

In June, 1896, the author had occasion to calculate the stresses, sections and weight of a three-hinged spandrel-braced parabolic arch, the outline and main dimensions of which are shown in Fig. 1. Shortly after finishing these calculations he determined to make the corresponding calculations for an arch of the same outline having two hinges, in order to determine their exact relative economy as regards weight of metal. It was not until recently, however, that the second calculation was completed. As it would seem that these results might be of some interest to members of this Society, they are here given, together with a brief exposition of the method used in calculating the stresses in the two-hinged arch.

These arches were designed for a live load of 2 160 lbs. per lineal foot or 36 000 lbs. per panel of the arch, and a dead load of 2 880 lbs. per lineal foot or 48 000 lbs. per panel of the arch. This unusually large dead load is due to a heavy asphalt floor carried by buckled plates.

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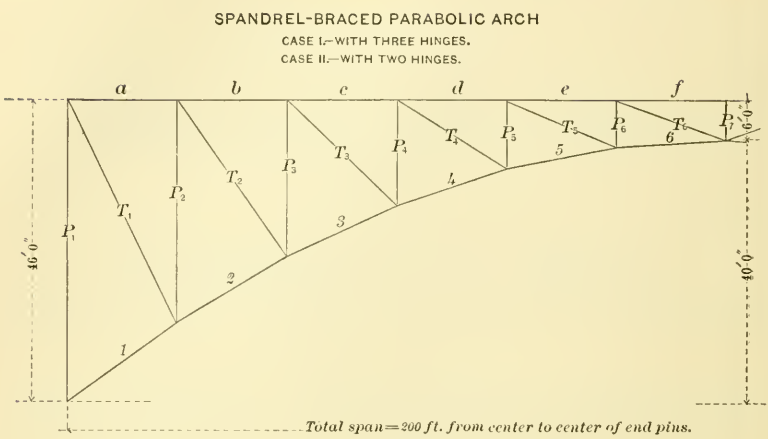

FIG. 1.

No metal less than $\frac{5}{16}$ in. in thickness was used for either case; and only two sizes of channels—12 and 15 ins.—were used for the members other than the arch ring. By introducing another shape, say 10-in. channels, a small saving of weight could have been made in both cases, but the appearance of the arch is better on account of the greater uniformity, and the construction is cheapened for the same reason.

The unit stresses used in proportioning the members were:

For tension

Live-load stresses, 11 000 lbs. per square inch.

Dead-load " 22 000 " "

For compression

In arch ring and top chord

Live-load stresses, $12\,000 - 55\,\frac{l}{r}$ lbs. per square inch.

Dead-load " $24\,000 - 110\,\frac{l}{r}$ " "

In web members

Live-load stresses, $11\,000 - 50\,\frac{l}{r}$ lbs. per square inch.

Dead-load " $22\,000 - 100\,\frac{l}{r}$ " "

It would have been more consistent and better to have used for the arch rings and top chords also the compression formulas used for

web members, as the loading producing maximum stress in these members is partial, not covering the entire span, for almost all the members of both cases.

These unit stresses, as will be recognized, are those for medium steel, of Theodore Cooper's Specifications of 1896 for Highway Bridges.

The temperature stresses in the two-hinged arch were treated as dead-load stresses. Members subject to alternate stresses of tension and compression were proportioned to resist both kinds of stress, with $\frac{8}{10}$ of the smaller added to either.

TABLE No. 1.—Case I. Stresses, Sections and Weights for a Three-Hinged Arch Having a Span Length of 200 ft. from Center to Center of End Pins.

Member.	Live-load stresses.	Dead-load stresses.	Sections.	Sq. ins.	Weight in pounds.
P_1	+ 59 000 / − 77 000	24 000	2 15″ channels	100 = 19.6	
P_2	+ 50 400 / − 86 400	48 000	2 12″ "	89 = 17.4	
P_3	+ 37 900 / − 73 900	48 000	2 12″ "	65 = 12.7	
P_4	+ 24 700 / − 60 700	48 000	2 12″ "	65 = 12.7	14 230
P_5	+ 11 700 / − 47 700	48 000	2 12″ "	65 = 12.7	
P_6	+ 30 600 / − 66 600	48 000	2 12″ "	65 = 12.7	
P_7	− 36 000	48 000	2 12″ "	65 = 12.7	
T_1	± 65 800	.....	2 12″ "	100 = 19.6	
T_2	± 61 600	.....	2 12″ "	80 = 15.7	
T_3	± 54 900	.....	2 12″ "	65 = 12.7	
T_4	± 51 100	.....	2 12″ "	65 = 12.7	15 560
T_5	± 49 600	.....	2 12″ "	65 = 12.7	
T_6	± 130 800	.....	2 12″ "	138 = 27.1	
a	± 29 200	.....	2 15″ "	96 = 18.8	
b	± 64 500	.....	2 15″ "	96 = 18.8	
c	± 103 100	.....	2 15″ "	96 = 18.8	14 230
d	± 134 300	.....	2 15″ "	124 = 24.3	
e	± 122 800	.....	2 15″ "	124 = 24.3	
f	.....	.....	2 15″ "	96 = 18.8	
1	− 334 800	446 400	{4 Ls. 4 × 4 × 48 / 2 pls. 24 × 11/16}	= 51.8	
2	+ 7 100 / − 321 900	419 800	{4 Ls. 4 × 4 × 44 / 2 pls. 24 × 11/16}	= 50.3	
3	+ 21 600 / − 319 600	397 300	{4 Ls. 4 × 4 × 48 / 2 pls. 24 × 5/8}	= 48.8	
4	+ 41 500 / − 326 200	379 500	{4 Ls. 4 × 4 × 44 / 2 pls. 24 × 5/8}	= 47.3	36 180
5	+ 53 700 / − 329 200	367 100	{4 Ls. 4 × 4 × 44 / 2 pls. 24 × 5/8}	= 47.3	
6	+ 10 600 / − 281 200	360 800	{4 Ls. 4 × 4 × 44 / 2 pls. 24 × 9/16}	= 44.3	
			Details (from shop drawings)....		34 800
				Total weight =	115 000

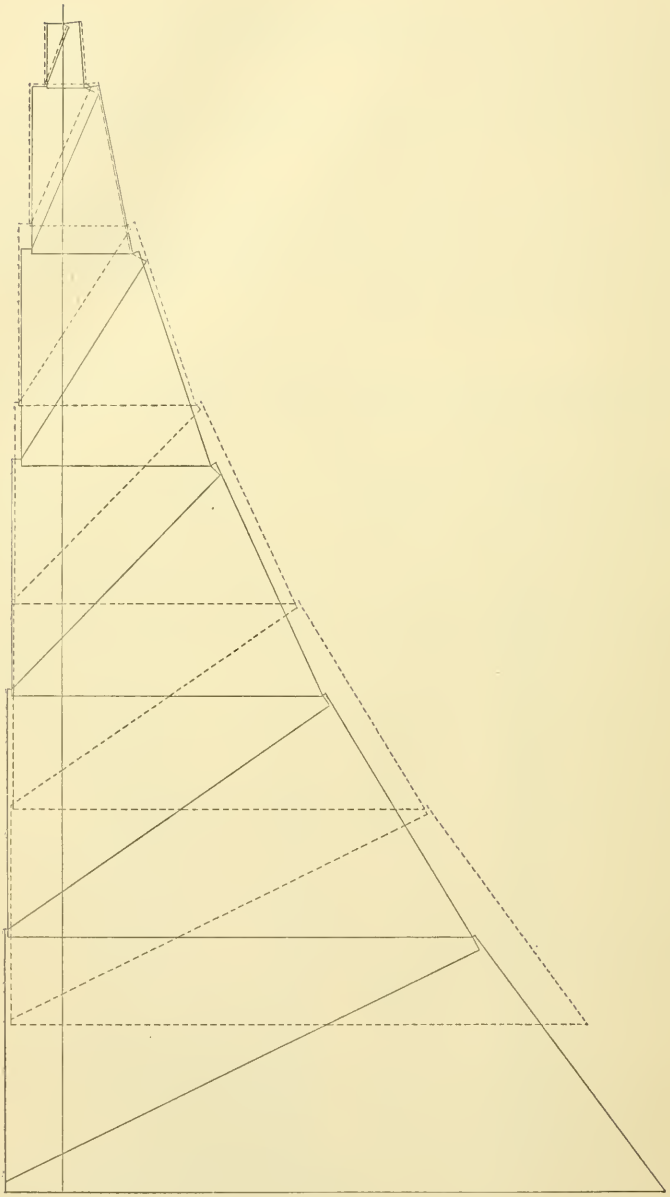

Fig. 4.

Case II.—The stresses for the two-hinged arch are not statically determinate, but may be obtained from the elastic properties of the arch.

The vertical components of the reactions for any load are the same as the reactions for a simple truss of the same span. The horizontal component of the reactions for any load can be found from the following formula:*

$$H = P \frac{\delta}{\delta'}$$

In which

H = Horizontal thrust produced by P;

P = Load at any point;

δ = Vertical deflection of the loaded point, due to a horizontal force of unity acting at the free hinge—one hinge assumed fixed and the other free—;

δ' = Horizontal displacement of the free hinge, due to a horizontal force of unity acting at the free hinge.

Fig. 2 will make the notation of this formula more clear than the definitions alone could.

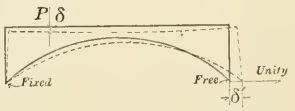

Fig. 2.

For simplicity of comparison the results are arranged as compactly as possible in Table No. 2.

The first column gives the marks designating the members (see Fig. 1). The second column shows the stresses due to a horizontal force of unity acting at one of the hinges. These stresses were very carefully figured analytically, and then checked graphically by means of the diagram, Fig. 3. From these stresses the changes in length λ of

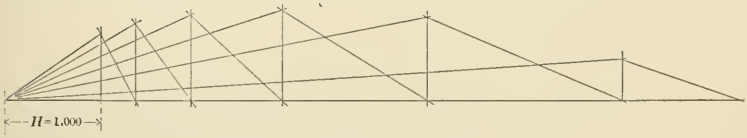

Fig. 3.

the various members were computed from the formula $\lambda = \frac{T L}{E A}$. In determining these values of λ, which are given in the third column, L was taken in feet, A, at present unknown, was taken equal to unity, and E was taken at 29 instead of 29 000 000, in order to give values of λ sufficiently large to allow them to be accurately plotted. These pre-

* "Roofs and Bridges," Part IV, Merriman & Jacoby.

liminary values of λ are 1 000 000 $\times$ A times their true value. Using these values of λ the displacement diagram shown in dotted lines in Fig. 4 was constructed. The full lines in this figure give the final displacement diagram, and from this diagram the ratio of δ to δ' was found to have the following values, beginning with the panel point at the end of the arch: 0.003, 0.203, 0.401, 0.590, 0.764, 0.904 and 0.962.

The thrust due to a load at the end of the span being so small, only 0.003 of the load, it is neglected in both the preliminary and final calculations.

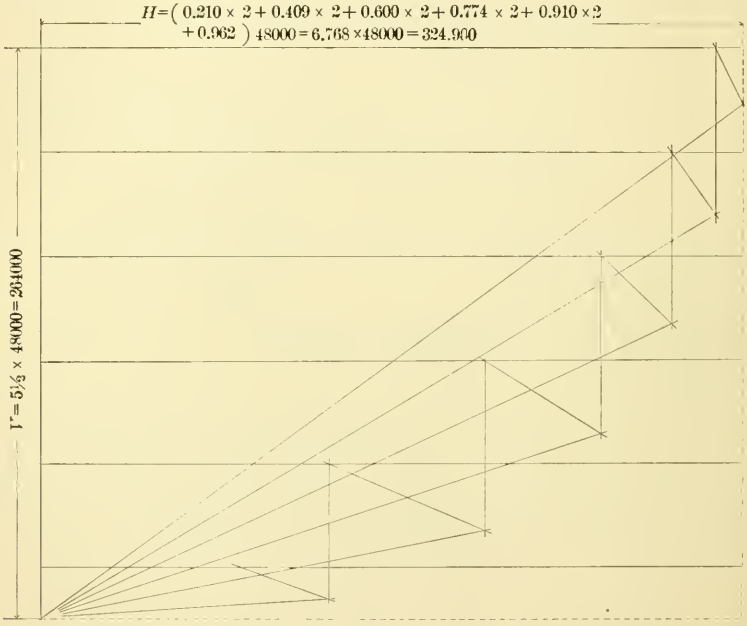

Fig. 5.

For full loading the preliminary value of the horizontal thrust is 6.686 $\times$ 48 000 lbs.; from this, by means of a diagram similar to Fig. 5, (Fig. 5 being the final dead-load stress diagram), the preliminary dead-load stresses are obtained. These stresses are given in the sixth column of the table.

Before finding the preliminary live-load stresses it will be well to find the stresses in the arch due to a vertical reaction of unity, supposed to be applied in this case at the left hinge. The stresses due to this load are given in the fifth column of the table. They were com-

puted analytically and checked by means of the diagram, Fig. 6. The upper figures in the fifth column, where two sets are given, are the stresses for the corresponding member in the right half of the arch.

To determine the greatest tensile and compressive live-load stresses to which any member of the arch is subjected, take 5 of the arch ring for example, and arrange the computations as follows:

$-H$		$+V$	
4.492×0.203		$13.019 \times \frac{1}{12}$	
$\times 0.401$	1.958	$\times \frac{2}{12}$	$\frac{5}{6} = 10.849$
$\times 0.590$		$\times \frac{3}{12}$	
$\times 0.764$	3.432	$\times \frac{4}{12}$	4.340
$\times 0.904$	4.061	$6.509 \times \frac{7}{12}$	3.797
$\times 0.962$		$\times \frac{6}{12}$	
$\times 0.904$		$\times \frac{5}{12}$	
$\times 0.764$	4.728	$\times \frac{4}{12}$	$2\frac{1}{3} = 15.208$
$\times 0.590$		$\times \frac{3}{12}$	
$\times 0.401$		$\times \frac{2}{12}$	
$\times 0.203$		$\times \frac{1}{12}$	

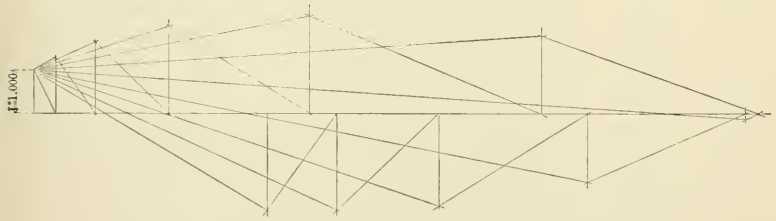

Fig. 6.

The value of H and V being those given in the second and fifth columns of Table No. 2 opposite 5, by inspection it can be seen that loads on the second, third, fourth and fifth panel points produce tension, and loads on the other points produce compression in the member 5. The computation of the stresses is now very simple, and we have:

For tension $(10.849 - 4.492 \times 1.958)\ 36\ 000 = 2.054 \times 36\ 000$
$= 74\ 000.$

For compression $(4.728 \times 4.492 - 15.208)\ 36\ 000 = 6.030 \times 36\ 000$
$= 217\ 000.$

The other live-load stresses were determined in this manner, and the computations were equally simple.

TABLE No. 2.—(*Continued*).

a ..	+ 0.362	+ 0.208	+ 0.011	− 0.493	− 14 500	− 12 700	+ 20 100 − 30 700	+ 20 800 − 30 200	3 600	2 15″ " 96 = 18.8	2 15″ " 96 = 18.8	
b ..	+ 0.935	+ 0.537	+ 0.029	− 7.009 − 1.402	− 36 500	− 33 200	+ 41 400 − 68 700	+ 43 000 − 67 500	9 400	2 15″ " 96 = 18.8	2 15″ " 96 = 18.8	
c...	+ 1.875	+ 1.078	+ 0.057	− 9.375 − 3.125	− 74 500	− 66 300	+ 61 000 − 115 500	+ 64 000 − 113 400	18 800	2 15″ " 96 = 18.8	2 15″ " 96 = 18.8	
d...	+ 3.404	+ 1.956	+ 0.059	−12.766 − 6.383	− 134 100	− 120 900	+ 66 500 − 166 300	+ 71 500 − 166 900	34 000	2 15″ " 169 = 33.2	2 15″ " 159 = 31.2	
e...	+ 5.469	+ 3.143	+ 0.095	−16.406 −11.719	− 214 700	− 192 000	+ 36 100 − 196 300	+ 44 200 − 188 300	55 000	2 15″ " 169 = 33.2	2 15″ " 159 = 31.2	
f...	+ 6.667	+ 3.831	+ 0.107	−16.667	− 260 900	− 232 800	− 195 400	+ 800 − 176 400	66 700	2 15″ " 183 = 35.9	2 15″ " 166 = 32.6	17 890
1...	− 1.240	− 0.884	− 0.018	+ 0.000	− 398 000	− 403 000	− 298 500	− 302 100	12 400	{4 Ls. 4 × 4 × 51 {2 pls. 20 × $\frac{3}{4}$ } = 50.0	{4 Ls. 4 × 4 × 53 {2 pls. 20 × $\frac{3}{4}$ } = 50.8	
2...	− 1.588	− 1.064	− 0.023	+ 0.575	− 357 600	− 364 000	+ 7 400 − 275 700	+ 7 000 − 281 200	15 900	{4 Ls. 4 × 4 × 51 {2 pls. 20 × $\frac{5}{8}$ } = 45.0	{4 Ls. 4 × 4 × 53 {2 pls. 20 × $\frac{5}{8}$ } = 45.8	
3...	− 2.135	− 1.354	− 0.033	+ 7.735 + 1.547	− 313 200	− 321 800	+ 23 200 − 258 600	+ 22 000 − 260 200	21 400	{4 Ls. 4 × 4 × 41 {2 pls. 20 × $\frac{5}{8}$ } = 41.1	{4 Ls. 4 × 4 × 43 {2 pls. 20 × $\frac{5}{8}$ } = 41.9	
4...	− 3.031	− 1.836	− 0.047	+ 9.882 + 3.294	− 259 900	− 272 900	+ 47 600 − 243 500	+ 44 900 − 249 700	30 300	{4 Ls. 4 × 4 × 41 {2 pls. 20 × $\frac{9}{16}$ } = 38.6	{4 Ls. 4 × 4 × 43 {2 pls. 20 × $\frac{9}{16}$ } = 39.4	
5...	− 4.492	− 2.633	− 0.083	+13.019 + 6.509	− 190 600	− 209 100	+ 74 000 − 217 000	+ 68 300 − 224 700	44 900	{4 Ls. 4 × 4 × 29 {2 pls. 20 × $\frac{1}{2}$ } = 31.4	{4 Ls. 4 × 4 × 29 {2 pls. 20 × $\frac{9}{16}$ } = 33.9	
6...	− 6.483	− 3.735	− 0.117	+16.442 +11.745	− 107 000	− 133 400	+ 72 000 − 152 500	+ 62 400 − 162 100	64 800	{4 Ls. 4 × 4 × 29 {2 pls. 20 × $\frac{1}{2}$ } = 31.4	{4 Ls. 4 × 4 × 29 {2 pls. 20 × $\frac{1}{2}$ } = 31.4	30 560
											Details (estimated)....	31 900
											Total weight =	109 000

*The Temperature Stresses in the Final Calculations were found to be the same as in the Preliminary Calculations.

TABLE No. 2.—Case II.—Stresses, Sections, Weights, etc., for a Two-Hinged Arch, Having a Span Length of 200 ft. from Center to Center of End Pins.

Truss Members.	Stresses due to H.	λ		Stresses due to V.	Dead-Load Stresses.		Live-Load Stresses.		Temperature Stresses.	Sections.		Weights of final sections in pounds.
		Prelim.	Final.		Prelim	Final.	Prelim.	Final.	Preliminary and Final.*	Preliminary.	Final.	
P_1..	+ 0.733	+ 1.162	+ 0.062	— 1.000	— 53 000	— 49 800	{+ 40 700 / — 80 300}	+ 42 000 / — 79 400	± 7 300	2 15″ channels 96 = 18.8 Sq. Ins.	2 15″ channels 96 = 18.8 Sq. Ins.	
P_2..	+ 0.817	+ 0.952	+ 0.051	{— 2.257 / — 1.296}	— 80 000	— 76 900	{+ 30 800 / — 90 800}	+ 32 000 / — 89 600	8 200	2 15″ " 96 = 18.8	2 15″ " 96 = 18.8	
P_3..	+ 0.903	+ 0.740	+ 0.044	{— 2.271 / — 1.654}	— 85 200	— 79 300	{+ 20 100 / — 82 500}	+ 21 400 / — 81 100	9 000	2 12″ " 85 = 16.7	2 12″ " 82 = 16.1	
P_4..	+ 0.958	+ 0.529	+ 0.039	{— 2.125 / — 2.042}	— 86 000	— 82 000	{+ 8 500 / — 72 700}	+ 10 500 / — 71 900	9 600	2 12″ " 70 = 13.7	2 12″ " 69 = 13.5	
P_5..	+ 0.881	+ 0.317	+ 0.025	{— 1.553 / — 2.277}	— 82 700	— 79 000	{+ 9 900 / — 71 600}	+ 10 700 / — 69 900	8 800	2 12″ " 65 = 12.7	2 12″ " 65 = 12.7	
P_6..	+ 0.431	+ 0.106	+ 0.008	{— 0.094 / — 1.781}	— 64 600	— 62 800	{+ 27 600 / — 76 200}	+ 28 100 / — 75 500	4 300	2 12″ " 65 = 12.7	2 12″ " 65 = 12.7	
P_7..	.....	.....	.....	.....	— 48 000	— 48 000	— 36 000	— 36 000	.....	2 12″ " 65 = 12.7	2 12″ " 65 = 12.7	15 220
T_1..	— 0.818	— 1.062	— 0.084	+ 1.115	+ 32 200	+ 28 800	{— 45 500 / + 69 500}	— 46 900 / + 68 400	8 200	2 12″ " 65 = 12.7	2 12″ " 65 = 12.7	
T_2..	— 0.998	— 0.999	— 0.079	{+ 2.756 / + 1.583}	+ 38 900	+ 35 300	{— 37 000 / + 66 000}	— 39 100 / + 65 400	10 000	2 12″ " 65 = 12.7	2 12″ " 65 = 12.7	
T_3..	— 1.304	— 1.039	— 0 082	{+ 3.279 / + 2.389}	+ 53 000	+ 45 500	{— 29 000 / + 67 200}	— 30 900 / + 65 100	13 000	2 12″ " 65 = 12.7	2 12″ " 65 = 12.7	
T_4..	— 1.805	— 1.224	— 0.097	{+ 4.002 / + 3.815}	+ 70 900	+ 63 600	{— 19 000 / + 72 000}	— 22 800 / + 70 300	18 000	2 12″ " 65 = 12.7	2 12″ " 65 = 12.7	
T_5..	— 2.245	— 1.403	— 0.095	{+ 3.958 / + 5.801}	+ 88 000	+ 78 300	{— 39 400 / + 105 600}	— 42 400 / + 101 400	22 500	2 12″ " 79 = 15.5	2 12″ " 75 = 14.7	
T_6..	— 1.273	— 0.778	— 0.033	{+ 0.277 / + 5.259}	+ 49 300	+ 43 000	{— 118 700 / + 155 700}	— 122 800 / + 154 200	12 700	2 12″ " 120 = 23.5	2 12″ " 121 = 23.7	13 430

Having determined the preliminary live and dead-load stresses, it now remains to determine the preliminary temperature stresses. From the displacement diagram, already constructed, the horizontal displacement of the hinge, due to a horizontal force of unity, is found to be $\frac{230.8}{1\,000\,000 \times a}$ ft. It will be assumed that the arch will be subjected to a range of temperature of 120° Fahr., or a variation of 60° from the mean. This is a less variation than is usually assumed, but as the metal of the arch under consideration is to be protected from the direct rays of the sun by a highway floor, it is probably enough. The change in length of the arch due to a variation of 60° Fahr., taking the coefficient of expansion as 0.0000065 per degree Fahr., is 0.078 ft. Dividing this change of length due to temperature by the change in length due to a thrust of 1 lb., we get $\frac{78 \times a \times 10\,000\,000}{1\,000 \times 2\,308}$ lbs. as the value of the temperature thrust. The value of a in this expression for the value of the temperature thrust is at present unknown; the average influence of the areas of all the members of the arch is very nearly represented by the influence of the area of a member of the top chord near the center of the arch. In this case the approximate value of the area of the member e, as determined from the preliminary live and dead-load stresses, is used as the value of a in the expression for the temperature thrust. This approximate determination of the area of e gives about 31 sq. ins., which gives for the preliminary temperature thrust $\frac{78 \times 31 \times 10\,000\,000}{1\,000 \times 2\,308} = 10\,000$ lbs. The preliminary temperature stresses are now determined by multiplying the stresses in the second column of the table by 10 000.

From these preliminary dead-load, live-load and temperature stresses the preliminary sections were determined.

Using these new areas, instead of unity used in the preliminary calculation, the final calculations of stresses are made in exactly the same manner as the preliminary calculations. It is found that the preliminary temperature stresses need not be changed for the final determination of the sections. A final determination of the sections shows only small changes from the preliminary. In most of the members there is no change, and the greatest change amounts to only 6%, and as the preliminary determination for this case was on the safe side it seems that the final calculation was hardly necessary. Where

an estimate only is required, the preliminary calculation would certainly be ample.

The calculations of weight give:

For Case I.	115 000 lbs.
For Case II.	109 000 "

This gives a saving in weight of $5\frac{1}{2}\%$ in favor of the two-hinged arch. A variation of 75° in temperature would have lessened the difference considerably, but the two-hinged arch would still have been lighter.

While it is hardly allowable to draw general conclusions from the consideration of a special case, it can be said that where an arch of this form (spandrel-braced) is suitable, the two-hinged arch is lighter than the three-hinged arch; it is also cheaper to construct, as there is no center pin and there are no adjustable members at the center of the arch. The floor system of the two-hinged arch would also be more simple than that of the three-hinged arch, for the great range of height of the center-pin of the three-hinged arch, due to temperature and live-load stresses, makes a troublesome break in the floor system at this point.

When spandrel-braced arches are used in series, supported on intermediate masonry piers, the two-hinged arch has the advantage of having less horizontal thrust, and therefore requiring smaller piers than the three-hinged arch. Great care must be taken in the construction of the masonry for the two-hinged arch, in order that it may not be subject to even slight settlement or displacement; but, taking this extra work into account, it is believed that the masonry for a series of two-hinged arches will cost less than the masonry for a series of three-hinged arches.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

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IMPACT TESTS OF STRUCTURAL STEEL.

BY S. BENT RUSSELL, M. Am. Soc. C. E.

TO BE PRESENTED OCTOBER 4TH, 1899.

In this paper will be given the results of some impact tests of small specimens of wrought iron and soft steel. The tests were made by a new method, under which the specimens were broken by tensile stress.

The matter to be presented will be given in its natural order, beginning with a few words on the need of tests of this character. The theory of the methods will then be given, followed by a description of the appliances used in the tests. The results of the experiments will then be stated, and, lastly, the conclusions that may be drawn from the observations.

Now, as to the need of study on the effect of impact on steel, if we leave out the engineers, the average man would be found to think that the strength of steel was its most important quality. Engineers, however, do not take the same view of the matter. To give an illustration, a well-known and experienced engineer once remarked to the author, that in ordinary engineering work he cared but little about the tensile strength of the steel he was to use, as compared with the general

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in *Transactions*.

reliability of the metal, *i. e.*, its uniform toughness and ability to withstand shocks and distortion.

This view is, without doubt, held by the majority of experienced engineers. In fact, it may be said, that among engineers the importance of shock resistance is generally admitted. Most engineers, however, will be found content with the methods now in use for determining the shock resistance of steel. They will contend that any skilful blacksmith can test a sample of steel and say whether it is tough and reliable or not.

There is, on the other hand, among engineers and scientific men, a rapidly growing feeling as to the importance of impact tests of structural materials, made, not with the blacksmith's methods and muscular sense, but in accordance with the stricter rules of experimental science. No argument is needed, then, to show that at this time studies of the effect of impact on steel are quite in order, and that the first step is to seek a satisfactory method of testing steel. The hope of finding a method which would be in advance of present practice has led to the experiments herein given.

Theory of the Methods Used.

The following description, with Figs. 1 to 5, shows how the new design may be evolved, step by step, from the old type of impact machine.

Fig. 1 shows in outline the familiar form of impact testing machine, where the test bar rests on two rigid supports and is struck in the middle by a falling weight. Owing to the rigidity of the knife-edges, *K*, the energy of the blow must be absorbed in deflecting the test piece *F*.

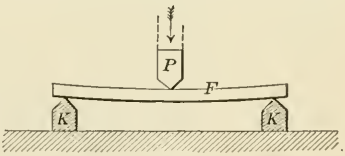


Fig. 1.

Suppose, now, a set of conditions where the bar *F* is comparatively rigid, and, in place of one of the knife-edges, there is a yielding support or spring *S*, as shown in Fig. 2. The energy of the blow from the falling weight *P* will now be absorbed in compressing the spring *S*. This will be strictly true only where the bar *F* and the remaining knife-edge *K* are perfectly rigid.

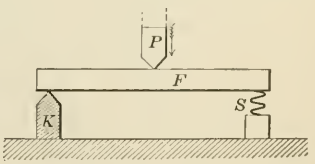


Fig. 2.

Fig. 3 shows the same arrangement, except that the spring S acts in tension and is supported by a bracket B, and, if the bar F, the knife-edge K and the bracket B are perfectly rigid, the entire energy of the blow from P will be absorbed in stretching the spring S.

In order to lessen the strain on the bar F and on the knife-edge K, the point of application of the blow may be changed, as shown in Fig. 4.

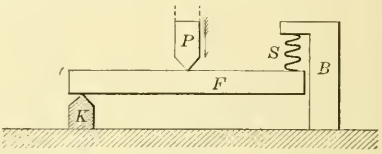


Fig. 3.

Now, if we substitute in place of the spring S a test bar of metal we have an impact testing machine which will show the tensile resistance to impact of the material.

Let us now make another change in the arrangement. Place the knife-edge K vertical and substitute for the falling weight a pendulum swinging on a horizontal axis above and parallel to the fork-bar F, so that the pendulum P will strike a horizontal blow on the bar F, as shown in Fig. 5, which is a ground plan. Let the knife-edge K and the bracket B rest against independent piers, so that, when the specimen breaks, the fork-bar F will swing to one side and allow the pendulum to pass.

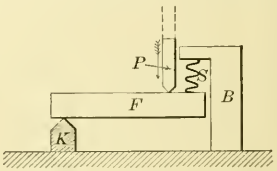


Fig. 4.

With this arrangement the pendulum may be raised to a given height and released. It will strike the bar F, tear the specimen in two and rise after the blow to a certain height. The difference between the height through which the pendulum fell and the height to which it rose after the blow shows the amount of energy absorbed in tearing the specimen. Fig. 5 may be said to show diagramatically the form of machine used in the experiments herein described. From the foregoing it will be understood that by this method the specimens are broken by direct tension, and hence the tests may be called tensile impact tests.

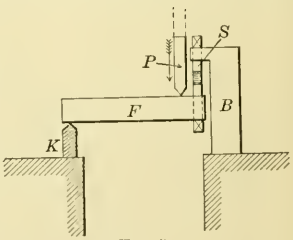


Fig. 5.

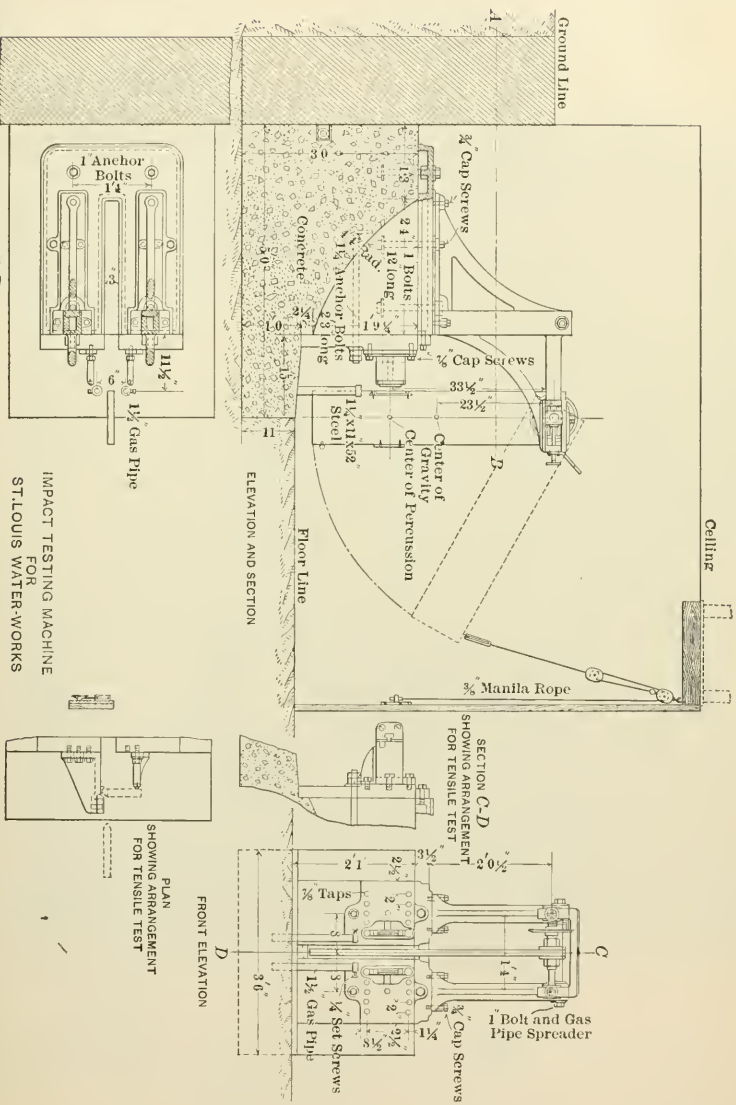

A
Ground Line
¾" Cap Screws
1" Anchor Bolts
14"
Concrete
1" Bolts
12" long
Cast Iron
1¼" Anchor Bolts
2' 3" long
⅛" Cap Screws
33½"
23½"
1¼"x1¼"x32" Steel
Center of Gravity
Center of Percussion
B
Ceiling
Floor Line
ELEVATION AND SECTION
⅜" Manila Rope
PLAN AT A-B
11½"
1½" Gas Pipe
IMPACT TESTING MACHINE FOR ST. LOUIS WATER-WORKS
SECTION C-D SHOWING ARRANGEMENT FOR TENSILE TEST
PLAN SHOWING ARRANGEMENT FOR TENSILE TEST
2' 1"
3½"
2' 0½"
⅛ Taps
D
FRONT ELEVATION
C
1½ Gas Pipe
¾" Set Screws
1¼
¾" Cap Screws
1" Bolt and Gas Pipe Spreader

Fig. 6.

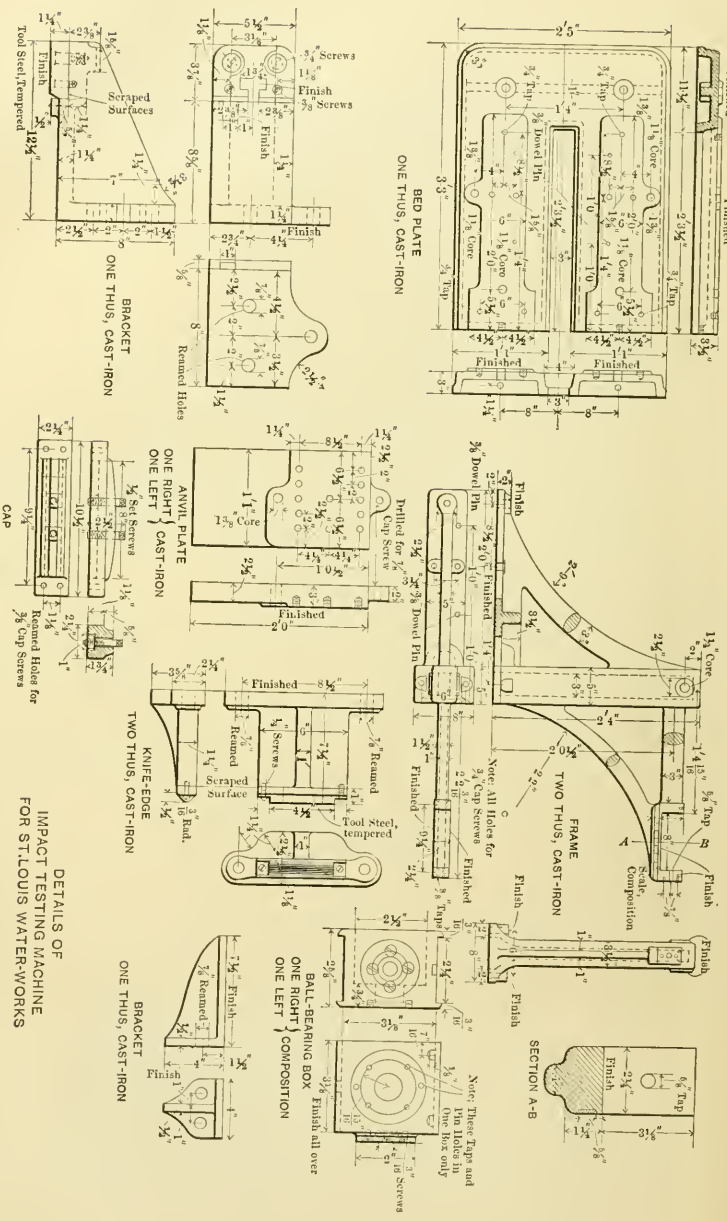
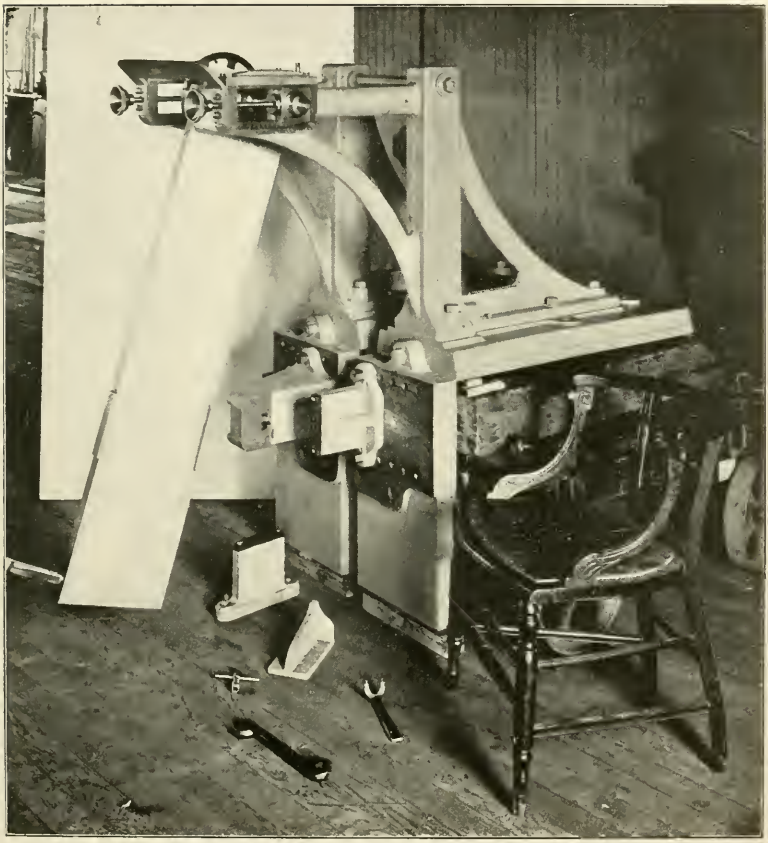


FIG. 7.

DETAILS OF IMPACT TESTING MACHINE FOR ST. LOUIS WATER-WORKS

PLATE VII.
PAPERS AM. SOC. C. E.
AUGUST, 1899.
RUSSELL ON IMPACT TESTS OF STEEL.



The Testing Machine.

The testing machine used in the experiments has been partly described and illustrated in the *Transactions* of this Society.* The original machine was devised by the author for making transverse tests by impact. The attachments by which tensile impact tests could be made were also devised by the author. Mr. William F. Schaefer rendered valuable assistance in the execution of the scheme. The attachments for tensile tests were made in March, 1898. Figs. 6 and 7 show the form and some of the details of a new testing machine now being built for the St. Louis Water-Works Extension. In principle, the two machines are alike.

The new machine will have a pendulum or hammer of forged steel, rectangular in form and weighing about 200 lbs. It will hang on a

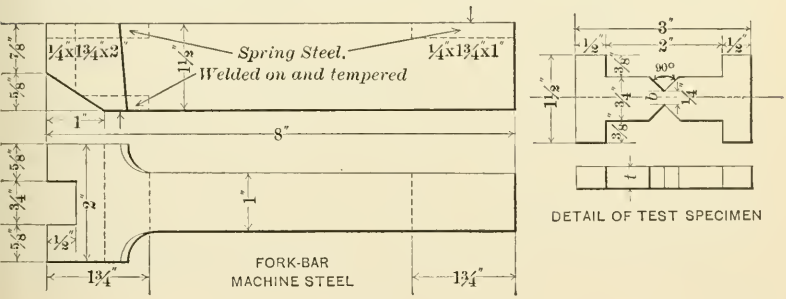


Fig. 8.

horizontal shaft resting in ball bearings. The striking edge of the pendulum and the fixed knife-edge will be of tempered tool steel.

The machine will be set up in a basement room, and the concrete anvil-block will abut against a stone cellar wall, below the level of the earth outside. By this arrangement the pendulum will swing clear of the floor of the room, and everything will be quite accessible and convenient. Plate VII shows the frame and mechanism as assembled in the shop.

Fig. 8 gives the dimensions of the lighter fork-bar used. One of the fork-bars was made of machinery steel with faces of tool steel welded on at all bearings. These faces were tempered and then ground true. The other fork-bar was made of tool steel throughout, and tempered and ground at the bearings.

* "Experiments with a New Machine for Testing Materials by Impact." By S. Bent Russell, M. Am. Soc. C. E. *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 237.

The bracket used is of cast iron. It is very heavy and rigid and is provided with a slotted plate of tempered steel. This plate receives the T-head of the specimen, and all bearing surfaces are ground true. There is a small shelf on the face side of the bracket which supports the fork end of the fork-bar when in position for a test. The other end of the fork-bar is held up by an adjustable support. When in position the axis of the fork-bar is on a level with the center of percussion of the pendulum.

The 7½ by 4-in. bracket, shown in Fig. 7, is used to support the heavy bracket while it is being bolted to the anvil plate.

In Fig. 6 two arrangements of the machine are shown. The first is for transverse impact tests, where two knife-edges are used.* The second is for tensile impact tests, where the bracket and fork-bar are used with one knife-edge only. In Plate VII the parts are shown as set up for tensile tests.

In building these machines great care is taken to insure rigid supports. A solid concrete foundation is used, a heavy anvil plate is carefully bedded against the concrete, the seats of the bracket and knife-edge are planed off true, and the tempered steel striking plates are ground true on the seat and rest against scraped surfaces.

This care is necessary in order that there may be no springing in the joints when the blow is struck. The rule is that all fixed parts must be heavy and rigid, and all joints perfectly fitted and firmly bolted.

The hammer used in the experiments weighs 103 lbs., or about half what the new hammer will weigh. The relative proportions of its dimensions are the same as in the new hammer.

It is important that the hammer should strike on its center of percussion, so that there may be no blow on the trunnions. Where the hammer is of simple form, the center of percussion is readily computed.† After the hammer is set up, the center of percussion may be verified by timing the oscillations, the period of which should be the same as that of a simple pendulum, whose length is equal to the distance from the axis to the center of percussion.‡

The center of gravity of the pendulum or hammer is found by trial before mounting, and, at the same time, the hammer should be

* *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 237.

† Johnson's "Materials of Construction," Art. 293.

‡ Rankine's "Applied Mechanics," Articles 544 and 607.

weighed, as the force of the blow is to be measured by the fall of the center of gravity in inches and the weight of the hammer in pounds. The pendulum is, of course, provided with an attachment for reading the height through which the center of gravity falls. The height through which the center of gravity rises after passing its lowest point is also shown.

We are now provided with a machine for breaking specimens in tension with a single blow. Before making actual tests, however, it is in order to consider what results will be obtained. In this paper the resilience of a specimen will be understood to mean its shock resistance or stopping power; that is, the amount of energy or work that will be required to rupture it. This energy will be expressed in inch-pounds.

If we raise the hammer 2 ins. and let it fall so as to break a specimen, and the hammer rises to a height of 1 in. after breaking the specimen, we would say that, as the hammer weighs 103 lbs., 103 inch-pounds of energy have been absorbed, so that the apparent resilience of the specimen is 103 inch-pounds. The loss due to friction in swinging is easily allowed for, therefore it may be considered that there is no friction.

In addition to breaking the specimen, there has been energy absorbed in other ways. The most important of these are the inertia and the springing of the fork-bar.

The Determination of Errors.

Problem: To Find the Loss due to the Inertia of the Fork-Bar.—According to Hodgkinson, the observed resilience is to the true resilience as $\frac{I + W}{W}$, when W is the weight of the hammer and I is the inertia of the test bar.* The problem is, then, to find I.

To simplify the problem we will assume that the fork-bar or the bar to be struck is a rectangular prism having its axis or fixed point at one end x, Fig. 9, and that it is struck at the other end. The length of the bar we will call r', and we will consider that the thickness and width of the bar are small enough to be neglected, or, in other words, that the bar is a straight line revolving about one end and having weight, but no width or thickness.

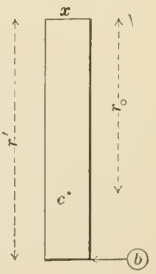


Fig. 9.

* *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 239.

Let W_1 = the weight of the bar.

Let us now substitute, for our rectangular hammer striking on its center of percussion, an equivalent simple pendulum. Let b equal the weight of this simple pendulum.

We now have the case of the familiar ballistic pendulum; treating the bar, whose length is r', as the ballistic pendulum, and the weight b as the projectile (see Fig. 9).

The formula for this case will be found in Rankine's "Applied Mechanics."* Taking the same notation as Rankine:

Let v = the velocity of the weight b before striking;

r_o = the distance from the axis x to c, the center of gravity of the weight and bar together;

W = the weight of the ball b and the bar together;

l = the length of a simple pendulum equivalent to the combined mass revolving about the axis x;

$B = \frac{W r_o}{l}$ = the portion of the joint weight to be treated as if concentrated at the center of oscillation, or a distance l from x;

V = velocity of the center of oscillation of the joint mass at the instant of perfect contact, when both bodies are moving at the same velocity;

g = the acceleration due to gravity.

Professor Rankine shows that $\frac{B\ V^2}{2\ g} = \frac{b\ v^2}{2\ g} \cdot \frac{b\ r'^2}{B l^2}$.

To find the value of $\frac{b\ r'^2}{B l^2}$:

Let W_1 = the weight of the bar alone;

Let I' = the moment of inertia of the joint mass = $(\frac{1}{3}\ W_1 + b)\ r'^2$;

Let $\rho^2 = \frac{I'}{W}$ = the square of the radius of gyration of the joint mass; $l = \frac{\rho^2}{r_o}$.†

Substituting and cancelling out we find

$$\frac{b\ r'^2}{B l^2} = \frac{b}{\frac{1}{3}\ W_1 + b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

*Article 607, p. 549. † Rankine, Art. 581.

which is the desired solution for the case of a hammer in the form of a simple pendulum.

To get this result in terms of known properties of the rectangular hammer: $b = \frac{W_2 R_2}{l_2}$.*

Where W_2 = weight of hammer;

R_2 = distance from axis of hammer to center of gravity of same; and

l_2 = distance from axis of hammer to center of percussion of same.

Substituting in (1) we get:

$$\frac{b r'^2}{B l^2} = \frac{1}{1 + \frac{W_1 l^2}{3 W_2 R_2}}; \text{ hence } \frac{B V^2}{2 g} = W_2 h \frac{1}{1 + \frac{W_1 l_2}{3 W_2 R_2}} \quad \ldots.(2)$$

= effective energy.

Where h = the fall of the center of gravity of the hammer.

Let $W_2 h = R'$, and $\frac{B V^2}{2 g} = R$.

Then from equation (2)

$$\frac{R'}{R} = \frac{1 + \frac{W_1 l_2}{3 W_2 R_2}}{1} = \frac{\frac{W_1 l_2}{3 R_2} + W_2}{W_2} = \frac{I + W}{W}$$

Where $I = \frac{W_1 l_2}{3 R_2}$ and W_2 is written as W, thus giving the value of I in Hodgkinson's rule.

Conclusion.—The inertia of the fork-bar may be taken as one-third of the weight of the bar by the ratio $\frac{l_2}{R_2}$, or the square of the ratio of the radius of the center of percussion to the radius of the center of gravity of the hammer.†

This rule is, of course, only approximately true for a bar of the shape actually used. The true correction for the bar could be found by the same line of reasoning, taking all the dimensions into account.

In the method above given no account is taken of the deflection of the fork-bar, which is assumed to be inconsiderable.

The rule may be applied to one of the fork-bars used in the experiments as follows:

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* Rankine, Art. 607.

† Compare Merriman's "Mechanics of Materials," Articles 103 and 111.

Weight of fork-bar No. 1 = 6.39 lbs.;

Weight of hammer = 103 lbs.;

$R_2 = 18$ ins.;

$l_2 = 25.75$ ins.

As $W_1 = 6.39$ ins., $\frac{W_1 l_2}{3 R_2} = \frac{6.39 \times 25.75}{3 \times 18} = 3.047 = I$; or the inertia of the bar in Hodgkinson's rule; and as

$W = 103$, $\frac{I + W}{W} = \frac{3.047 + 103}{103} = \frac{102.96}{100}$; hence $\frac{W_2 h}{R} = \frac{102.96}{100}$; or the total energy absorbed is nearly 3% greater than the resilience of the specimen, the difference being due to the inertia of the fork-bar.

As the error is directly proportional to the weight of the fork-bar, we would have with a fork of half the weight an error of less than 1½ per cent.

With any hammer of similar proportions, where the weight of the fork-bar is less that one-tenth of the weight of the hammer, the error due to the inertia of the fork-bar will be less than 5 per cent.

Problem: *To Find the Error due to the Spring of the Fork-Bar.*—We will consider the fork-bar as a beam supported at the ends and carrying a concentrated load. We first find the load P, which is determined by the maximum pull on the specimen at S_1 and the relative distances $S_2 P$ and $P S_1$, Fig. 10. From the dimensions of the beam and the modulus of elasticity of the metal we may find the deflection y of the beam at the loaded point. With all dimensions given, we find that by the accepted formulas the load P is directly proportional to the reaction S_1 and the deflection y is directly proportional to P. Hence y is directly proportional to S_1 or $y = K' S_1$ where K' is constant.

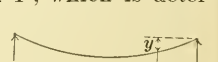

FIG. 10.

The energy absorbed in producing this deflection is

$$R' = \frac{P y}{2} = \frac{K'' S_1 y}{2}.$$

Where K'' is constant, substituting for y we get $R' = K S_1^2$. Where K is constant, which means that the loss of energy in springing the fork-bar is proportional to the square of the maximum pull on the test specimen, K may be readily computed for given conditions by well-known formulas.

In this way the value of K for the light bar used in these experiments has been computed to be equal to 0.0000001106.

With this value, taking a specimen having an ultimate strength of 10 000 lbs. for example, we find that the springing of the bar will absorb the energy: $R' = K S_1^2 = 11.06$ lbs., where $S_1 = 10\,000$ lbs. If the supposed specimen absorbed 400 inch-pounds in breaking, we would have an error of $\frac{11.06}{400} = 2.76$ per cent.

This may be considered an average case. If the specimen should have half the strength and half the resilience above given we would have half the percentage of error, or 1.38 per cent.

As a stiffer bar may be used with stronger specimens, it would seem that with metal of ordinary strength and ductility there should be no difficulty in keeping the proportionate error below 5%, and that without using so heavy a fork-bar as to involve a large error from inertia.

TABLE No. 1.—Resilience of Wrought Iron—Effect of Decreasing the Weight of the Fork-Bar.

Kind of wrought iron.	Resilience in Inch-Pounds per Square Inch of Section at Nick. R_2		Gain, per cent.
	With fork-bar weighing 6.39 lbs.	With fork-bar weighing 3.85 lbs.	
Tennessee common.....	2 115	2 165	2.4
Tennessee charcoal.....	2 885	3 046	5.5
Average gain.....	.....	.....	3.95

Note.—All the specimens of each metal were of the same form and dimensions, and were cut from the same bar. Each value of R_2 is the average of six tests.

Table No. 1 gives the results of experiments made to learn the effect of decreasing the weight of the fork-bar when other conditions are kept as nearly uniform as may be. The results show that the energy absorbed in breaking a specimen is greater with the light bar. This would seem to indicate that the error due to the springing of the fork-bar is greater than the error due to the inertia effect. The difference, however, may be due to variations in the metal.

Without further investigation, it seems fair to presume that the inherent errors of this method of testing need not exceed 10%, or, in other words, over 90% of the blow should be absorbed by the specimen itself.

In making comparative tests with the same fork-bar, the error due

to inertia is of no effect. In the results of the experiments no correction has been made for either the inertia or spring of the fork-bar. It is thought that, were all the corrections made, the comparative values would not be changed materially.

Preparing Specimens.

Taking up first the design of the specimen bars, we find that on account of the great toughness or resilience of steel, it is necessary to test but a small volume of the metal. As it is difficult to confine the effect of a blow to a definite volume of metal, a nicked section was adopted, a standard nick of simple form being used.

In Fig. 8 is shown the dimensions of the specimen bars used. As they are all cut from flat sections, all the dimensions are constant, except the thickness t.

To allow for the varying thickness, the resilience of a specimen is divided by the area of the cross-section at the nick, and the result is called the resilience per square inch. The results are believed to be comparable.

It should be remembered, of course, that a test specimen of different form would not give the same resilience per square inch. Objection may be made to the use of any particular form of nick, but it is believed that tests made with the form adopted give a good idea of the quality of the metal. The blacksmith judges a bar of steel by nicking it and breaking it with his hammer and anvil. We are simply improving on his method by using a nick made to gauge, and measuring the energy of the blow.

It is, perhaps, in order to note here that in the *Transactions* * of the Society will be found the results of previous experiments made by the author, in which nicked specimen bars of steel were broken by a single blow. In these previous experiments the bars were broken transversely. The great range in the values of resilience of steels tested in this manner led the author to think that transverse breaking gave an unfair advantage to the softer steels. Hence, this study of impact tensile tests with nicked specimens of steel.

The specimen bars used in the experiments were cut to shape in a milling machine, as experience showed this to be the best way to secure sharp and uniform nicks. The milling cutters were ground in a

* *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 237.

universal cutter and tool grinder, so as to secure a perfect cutting tool. A number of specimens are usually cut together on the milling machine.

Breaking Specimens.

The specimens are numbered and then calipered with a micrometer. A specimen bar is then placed in the bracket slot and the fork-bar adjusted. The hammer is raised to a given height and released. It falls, strikes the fork-bar and breaks the specimen. The height to which the hammer swings is recorded. The proper correction for friction of the hammer is noted, and the observer is ready for another specimen. Table No. 2 shows the form of record kept.

TABLE No. 2.—Impact Test.—Office of Water-Works Extension. —St. Louis, October 27th, 1898.—Specimen of Wrought Iron (Tenn. Char.) Taken from Iron Company.—Tested for Tensile Resilience, with Results herewith Appended.

Lot No. 4. Weight of fork-bar, 3.85 lbs.

Experiment No.	Melt No.	Bar No.	Initial fall of hammer, in inches. F.	Rise after blow, in inches. S.	Correction for friction, in inches. C.	Thickness of plate, in inches. t.	Width at nick, in inches. b.	Area, in square inches at nick. $A = tb$.	Effective fall of hammer. $H = F - (S + C)$.	Total resilience, in inch-pounds. $R = 103\ H$.	Resilience per square inch, in inch-pounds. $R_2 = \frac{R}{A}$
1 112	*B*	1	1.80	0.13	0.03	0.231	0.230	0.05313	1.64	168.92	3 179.
1 113	*B*	2	1.80	0.21	0.03	0.231	0.230	0.05313	1.56	160.68	3 005
1 114	*B*	3	1.80	0.18	0.03	0.233	0.230	0.05359	1.59	164.77	3 074
1 115	*B*	4	1.80	0.17	0.03	0.232	0.231	0.05359	1.60	164.80	3 075
1 116	*B*	5	1.80	0.29	0.03	0.232	0.230	0.05336	1.48	152.44	2 856
1 117	*B*	6	1.80	0.18	0.03	0.232	0.230	0.05336	1.59	164.77	3 087
										Average =	3 046

TABLE No. 3.—Resilience of Wrought Iron—Effect of Increasing the Initial Fall of the Hammer.

Kind of wrought iron.	Initial fall of hammer, in inches. F	Number of tests made.	Resilience, in inch-pounds per square inch of section at nick. R_2
Tennessee common.....	2	3	2 524
	4	3	2 488
Tennessee charcoal.....	2.5	3	3 549
	4.5	3	3 748

Note.—All the specimens of each metal were of the same form and dimensions, and were cut from the same bar.

TABLE No. 4.—Nicked Iron and Steel Bars.—Resilience per Square Inch.

Bars nicked as shown in Fig. 8.

Metal.	Lot No.	Number of Melt.	Number of tests made.	Thickness of plate, in inches, t.	Resilience, in inch-pounds per square inch of section at nick. R_2.	Ultimate tensile strength, in pounds per square inch.*	Percentage of elongation in 8 ins.*
Iron, Norway.....	2	.....	8	0.25	7 535	41 500	28.2
" Tenn. common.	3	A	6	0.25	2 506	55 000	21.2
" " charcoal.	4	A	6	0.25	3 648	52 500	27.5
" " " .	4	B	6	0.25	2 885	.....	....
" " common.	3	B	6	0.25	2 115	.....	....
" common	5	A	6	0.25	2 176	50 700	14.2
" "	6	B	6	0.31	3 290	50 000	23.1
" "	6	C	6	0.37	2 140	52 500	18.1
" "	6	D	6	0.44	1 640	57 200	15.6
" "	6	E	6	0.50	3 500	54 800	19.6
" "	6	F	6	0.56	2 050	54 800	18.4
" "	6	G	6	0.62	2 030	52 800	19.0
Soft steel (plates)....	6	743	2	0.32	7 657	53 750	30.3
" " "	6	743	2	0.44	7 600	54 800	29.6
" " "	6	749	2	0.44	9 028	57 000	31.5
" " "	6	757	2	0.32	7 645	56 900	30.5
" " "	6	757	2	0.44	6 100	55 600	27.7
" " (angles)...	11	A	6	0.32	7 290	62 200	31.0
" " "	11	B	6	0.37	1 550	54 500	31.5
" " "	11	C	6	0.50	2 950	64 800	29.1

* The values in these columns are the result of one test only.

Table No. 3 contains the results of a few experiments made to show the effect of increasing the initial fall of the hammer. Apparently the effect is small compared with the variations in the metal.

Table No. 4 gives the values of resilience found with a number of samples of wrought iron and soft steel. In the last two columns are given for comparison the strength and ductility of each melt as shown by the ordinary tensile tests with the load applied gradually. The thickness t is in all cases the thickness of the plate or bar as rolled.

The steel of Lot No. 6 was basic open-hearth steel. The specimens were cut from large plates. The tests showed very uniform material. The steel of Lot No. 11 was said to be medium steel, but the tensile tests indicated that it should be graded as soft steel. These specimens were cut from angles about 3 by 3 ins. in size. These tests showed a lack of uniformity in the material. All the wrought-iron specimens were cut from bar iron.

On examining the values given in Table No. 4 we note that the highest value obtained with wrought iron is 7 535 inch-pounds per square inch, with ¼-in. Norway iron. The lowest value obtained with

wrought iron of the same thickness is 2 115 inch-pounds. Taking all thicknesses of wrought iron we get a range of from 1 640 to 7 535.

The highest test of steel is over 9 000 inch-pounds and the lowest is 1 550 inch-pounds.

In Melt No. 743 we get practically the same results with metal of different thicknesses, while in Melt No. 757 the thicker metal makes a poorer showing. This may be due to the thicker plate having been finished in the rolls at too great a heat.

A study of all the values given in Table No. 4 indicates that the resilience per square inch is not proportional to the ultimate strength, nor to the proportionate elongation, nor to the product of the two. We note, too, that the proportionate range in value is greater for the resilience than for either ultimate strength or elongation.

Conclusion.

In conclusion we may review the work briefly. The tests given were made by a new method of breaking small specimens in tension by impact. We find that there are two important errors which may be said to be peculiar to the method of testing. We find the first of these or the error due to inertia, theoretically determinate. We find that the second of these errors or that due to the spring of the fork-bar is determinate to the degree that the tensile strength of the specimen is known. The other errors in the method are those common to all impact tests. We find that the results obtained are determined by the form of the specimen, and are hence only comparative.

We find that the tests that have been made by this method indicate that the resilience or shock resistance of rolled steel cannot be predicted from the tensile strength and elongation.

The values obtained will of course have but little other meaning until they have been interpreted by experience and by further experiment. It is suggested, however, that in time, tests of this sort may become a valuable aid in judging and recording the quality of structural steel.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

WHEEL CONCENTRATIONS AND FATIGUE FORMULAS IN BRIDGE DESIGN.

An Informal Discussion at the Annual Convention, June 27th, 1899.*

SUBJECTS FOR DISCUSSION:

"Should the use of the Method of Wheel-Concentrations be discontinued in determining the Stresses in Railroad Bridges?"

"In view of present knowledge of the Effect of Repeated Applications of Load, should Fatigue Formulas be used in Bridge Design?"

By Messrs. GUSTAV LINDENTHAL, PALMER C. RICKETTS, W. A. DOANE, GEORGE S. MORISON, W. L. COWLES, E. M. SCOFIELD, HENRY B. SEAMAN, J. P. SNOW, EDWIN THACHER and J. A. L. WADDELL.

Mr. Lindenthal.

GUSTAV LINDENTHAL, M. Am. Soc. C. E.—The objection to the method of wheel concentrations for finding the strains in bridge members does not arise so much from the greater labor of calculation, but because the assumption of wheel loads as resting on knife-edges is a false and unnecessary refinement, for, in any case, the rails and ties intervening between the wheels and the bridge structure distribute the concentrated loads upon the length of the wheel base. It may be correctly assumed, for the bridge structure, that the loads rest on skids, instead of on wheels; the rails representing the skids. The strains in the floor and in the trusses are thus more accurately found, for the train-load plus the excess load of the locomotive, than by the method of wheel concentrations, which should be discontinued.

*The discussion of this subject, for which no formal paper was presented, is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to September 23d, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

Mr. Lindenthal.

There are indications that the growth of weight in locomotives will soon come to an end; it will be determined, not by the strength of rails, bridges or drawbar heads, which can be increased at will, but by the hardness of steel in wheel tires and rails, necessary to resist the wear and tear of the heavy engines. It will reach the limit when steel in the railhead would be crushed, or, if made still harder, would be too brittle for safety. It may be fairly assumed that 120 tons on a wheel base of 15 ft. is the practicable and economic limit of weight concentration on railroad track. This represents a quiescent load of 16 000 lbs. per lineal foot, corresponding to 30 000 lbs. per wheel; or from 60 000 to 75 000 lbs. pressure per square inch on the wheel tire and rail, which is close to, or exceeding, the elastic limit of hard steel.

Some rails have, indeed, been made of hard nickel steel, which may safely resist greater pressures, but they cost about double as much as ordinary rails. They are used only for the outside of curves, and, therefore, do not determine ordinary service conditions.

Fatigue formulas in bridge designing made their first appearance in American bridge specifications about fifteen years ago, mostly in the form of the Launhardt and the Launhardt-Weyrauch formulas, or modifications thereof. The previous practice, which is also still largely followed (and sometimes combined with the fatigue formulas) consisted in the variations of the unit stresses for different bridge members, depending upon their position in the bridge, using the lowest unit stresses in the floor system and the highest in the chord members of the trusses. This was done according to the judgment of the designing engineers, out of which gradually had grown certain standards. The exploitation of the fatigue formulas was probably prompted by a desire to bring the arbitrary but well-founded standards, as regards unit stress, under a uniform scientific rule, although that expectation was not fulfilled.

As every one knows, the formulas were deduced by Professors Launhardt and Weyrauch, who were not themselves the experimenters, from certain tests for the Prussian Government, made by Wöhler and Spangenberg about thirty years ago, with steel and iron bars, which were subjected to rapid repetitions and reversions of stress.

The formulas were regarded at the time as examples of elegant mathematical reasoning, and their plausibility won many believers. Nevertheless, their application to bridge designing is absurd, because the conditions on which they are based do not obtain in bridges. This is well recognized by European engineers, who rarely, if ever, use them.

Wöhler strained his iron and steel close up to and over the elastic limit at the rate of 5 to 20 repetitions per second. These conditions may sometimes occur in machinery, car axles, connecting rods, rails, and other like structures, where the tensional and compressional strains are nearly always the effect of sudden and long-continued inter-

Mr. Lindenthal. mittent forces, and complicated with bending, shearing, or torsional strains. But, even for them, the correctness of the formulas is open to serious question when the stresses are kept far below the elastic limit.

Steel rails, made of 80 000-lb. steel, for instance, and subject to fiber stresses from + 27 000 to — 27 000, and higher, from fast and frequent trains, are found to be perfectly safe. They wear out, but they do not break down.

The strains in iron and steel bridges, however, are of an entirely different character. They are primarily static, and in well-designed bridges rarely exceed half the elastic limit of the material. They are not complicated with other strains, as in machinery designing; and, moreover, the maximum strains occur only at long and rare intervals, and in cumulative form.

It surely is not proven, that slow repetitions and reversions of stress, far within the elastic limit of the steel and iron, affect in any way their durability. There is not a single authentic fact or experiment known to support such a contention. Wöhler, who was well qualified to scientifically interpret his own tests, never made such a claim himself.

The values of the Launhardt-Weyrauch formulas agree with Wöhler's tests only partially, within certain limitations of stress. But there are no facts and no tests to prove their applicability to stresses one-half or one-third as low as those that Wöhler used. In the nature of things, such experiments cannot be made. How many thousand years would it take to subject a steel bar of 1 sq. in. to reversions of stress from 12 000 lbs. in tension to 8 000 lbs. in compression, at the rate of one reversion per minute, and at intervals of one hour, before it would suffer in strength, if it ever would? This is assuming, indeed, a higher strain, a quicker reversion and a shorter interval than actually occur in the chords of many existing cantilever and arch bridges, dimensioned according to fatigue formulas. The waste of material which they imply, particularly for members subject to reversion of stress, is not very important for ordinary spans; but for long-span bridges, where the maximum strains occur only under extraordinary and rare conditions, it is a very serious matter, as the bridge is simply loaded down with useless material, adding to its cost, but not to its strength or durability. In the speaker's practice he has never used them, being convinced that the old English practice, also endorsed by Rankine, of separating the strains from dead and live load, and determining the cross-section for both, by using larger unit stresses for the former, and smaller unit stresses for the live load, and then adding the results, is founded on the most rational basis thus far conceived. With the great variations of load conditions and material in steel bridges, it is more than doubtful if the judgment, in that respect, of the experienced designer can ever be replaced in practice with any one or more formulas.

Palmer C. Ricketts, M. Am. Soc. C. E.—Some years ago in discussing the paper "Some Disputed Points in Railway Bridge Designing,"* the speaker predicted that the method of wheel concentrations would be abandoned in a very few years. It is passing into disuse, and that it has not disappeared long ago is to be attributed rather to the conservatism than to the common sense of bridge engineers. It should be abandoned, not because the calculations consume more time and are more difficult than by other methods, but because the method is unscientific, and its accuracy is only apparent. It is irrational and unscientific to obtain stresses by carrying to such extremes calculations which take no account of dynamic effects, while the actual errors in design due to ignorance of dynamic effects are probably many times greater than those due to differences in methods of obtaining stresses. In other words, such calculations will be unnecessary refinements as long as we have to guess at allowed unit stresses. Mr. Ricketts.

The subject of fatigue formulas and their use in bridge design was so fully covered in the paper "The Launhardt Formula, and Railroad Bridge Specifications,"† that it is almost unnecessary to discuss it now. About 80% of those who took part in that discussion were of the opinion that fatigue formulas were wholly irrational, and for that reason should not be used.

W. A. Doane, M. Am. Soc. C. E.—The stresses in the various parts of railway bridges can be more easily calculated, and with fully as close an approach to the strains that will actually exist in the structure after completion, by using the method of uniform live loads, rather than by taking certain live loads, which may not obtain with one locomotive out of ten. Mr. Doane.

The speaker is, therefore, in favor of assuming uniform loads in bridge design, the more so, perhaps, since the general advent of 80-lb. or 90-lb. rails helps materially to distribute the wheel loads on the track stringers, or on the plate floor, as the case may be.

George S. Morison, Past-President, Am. Soc. C. E.—There is one idea which the speaker has thought over for a good many years. He has always felt that the use of locomotive wheel-base concentration was radically wrong. That it will give results which are not materially bad is certainly true, but the principle of it seems to be very far from what should be followed. It is, briefly, that we take a piece of rolling stock of special weights at special distances and make that a basis for a structure which we expect to last 50 years, and which probably will last 25; and yet that same locomotive will be superseded in $2\frac{1}{2}$ years. Mr. Morison.

*By J. A. L. Waddell, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xxvi, p. 77.

†By Henry B. Seaman, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xli, p. 140.

Mr. Morison. The original idea has been somewhat improved by using typical locomotives. The specifications of 20 or 25 years ago contained locomotives with the weight on the wheels carried down to the nearest 10 lbs., and the distances between wheels given to the nearest inch—the speaker thinks that some of them went to fractions, but trusts that this was not really the case. The speaker has felt that this was entirely wrong, and a good many years ago he adopted a method which he has always used in drawing his specifications, and which he still thinks gives better results than any system of wheel-base loads.

There is one feature of the wheel-base load which, if drawn out on a diagram, shows its defects. It does not give a uniform line gradually progressing for the varying length of spans, etc., but a line of slight undulations. Now, those undulations ought, in some way, to be eliminated, and something should be obtained which would give a gradual rate of progression from a very short span up to a span of such length that a uniform load can be taken throughout. There are really four different things to be provided in selecting the loads for which railroad bridges are to be proportioned. They are: (1) a heavy wheel-base load; (2) a moderate load for greater lengths; (3) a gradual transition from the heavy wheel-base load down (if measured by weight) or up (if measured by length) to the lighter uniform load; (4) provision for an excess in those members most affected by moving loads.

To these, perhaps, one other should be added. It is simply the provision that for extremely short spans, those less than wheel-base loads, provision should be made for an excessive concentration, and that load ought to be very excessive; it has been the speaker's practice to consider that it was possible that the whole 20 wheel-base loads could be concentrated on two axles, or, perhaps more properly, that half of it could be concentrated on one axle.

It has also seemed expedient to have these provisions governed by coefficients rather than by absolute amounts, and to adapt different structures to loading of different weights, by varying the unit load to which the coefficients are applied.

In other words, there should be a table of coefficients, or a rule for determining the coefficient, for each length of span. Bridges should be classified for the duty they have to perform, and, by applying those coefficients to the unit weight, the weight which is to be used in calculating every length of span could be ascertained at once.

Following this general idea, the speaker adopted, and this was done about 1882, 20 ft. as the wheel-base length and 120 ft. as the length of two locomotives. He then adopted, as the uniform load for 120 ft. and all greater lengths, a weight per lineal foot varying from 2 000 to 4 000 lbs., according to the class of the structure. On the wheel-base length of 20 ft. this unit load was doubled. The transition

was effected by adding 1% to the uniform load for each foot of length of structure less than 120 ft., this increase becoming 100% at the wheel-base length of 20 ft. Impact and irregular strains must be provided for chiefly in those members in which variations are rapid and reversals are liable to occur, that is, in the counter members of the web, and to a varying extent in the main members of the web. This extra provision was then made by considering what portion of the strain in any web member would exist under a load equal in intensity to the moving load, but uniform throughout the span. Deducting this from the maximum strain in such member left a remainder which varied from nothing at the end of the span to the full counter strain at the center, and which might be called the variable effect. This variable effect was then estimated on a basis of $1\frac{1}{2}$ times the regular moving load, and gave results which, from their uniform increase from the ends to the center and their general character, seemed to conform with the proper ideas of proportioning a structure. Mr. Morison.

On this basis nearly every bridge the speaker has built for the last 16 years has been designed. There are, however, some defects. If applied on the basis of the length of the span, the load for the shearing

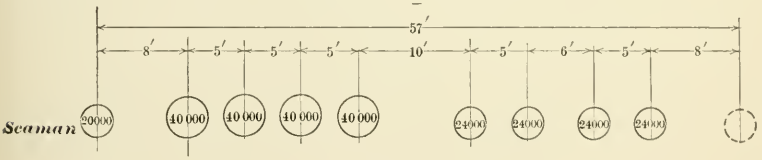


FIG. 1.

strains is too low; if applied on the basis of half the length of the span the load for the shearing strains is unnecessarily high. It also gives excessive results for lengths of span midway between 20 and 120 ft.; that is, the transition is made on a straight line from 20 to 120 ft., and it should be made on a curve. This objection has led to the study of a modification of this general scheme which will be shown in the following pages.

The calculations as given are applied to only one unit weight of train, and this has been taken at 5 000 lbs. per lineal foot, which is believed to be the highest unit weight now in use. The result, however, as obtained, is a series of coefficients which can be applied to any unit weight, which unit weight would probably, under present practice, be made to vary from 3 000 to 5 000 lbs.

Take as an illustration of a standard typical locomotive Fig. 1, used in a recent paper in the *Transactions* of this Society. This engine measures 57 ft. between corresponding points, when coupled up in a train of locomotives. The total weight is 276 000 lbs. This weight has been materially exceeded by recent engines, the two heaviest reported being a consolidation pusher on the Lehigh Valley Railroad,

Mr. Morison. and a consolidation engine running on the Union Railway of Pittsburg, full descriptions of both of which have been given in several of the technical papers. The length is somewhat less than that of the modern consolidation engine, which is usually somewhat over 60 ft., the total length of the Lehigh Valley engine being 62 ft., and that of the Union Railway engine exceeding 65 ft.

For purposes of simplicity it would seem advisable to adopt an engine 60 ft. long and to avoid the odd weight on the truck wheels and irregular distances between wheel bearings. These conditions are met by the engine shown in Fig. 2.

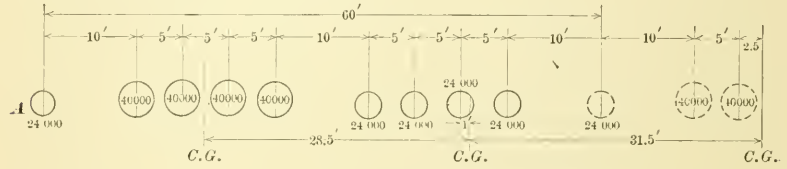


Fig. 2.

It will be observed that the only changes from the engine shown in Fig. 1 are the addition of 4 000 lbs. to the truck wheel and an addition of 2 ft. to the distance between the truck axle and the forward driving axle and the reduction of 1 ft. between the two central tender axles. This engine is 60 ft. long. It weighs 280 000 lbs., 4 000 lbs. more than the other. Every one of the weights is divisible by 4 000 and every distance is a multiple of 5; and yet the strains from this engine would differ so little from those of the engine shown in Fig. 1 that they would not be noticed in any structure that would be built.

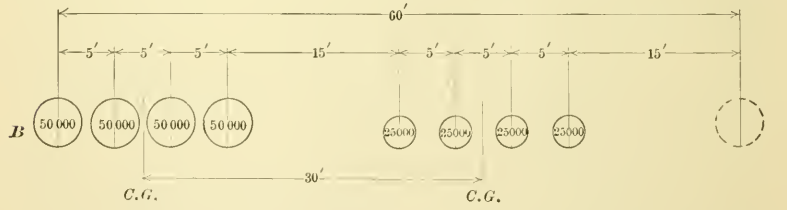


Fig. 3.

The distance between the center of gravity of the driving wheels and the center of gravity of all other wheels is 28.5 ft. A change in this dimension of 1.5 ft. would produce an engine in which the distance from the center of gravity of the four pairs of driving wheels to the center of gravity of all other wheels is half the length of the engine. It will be observed that the truck wheels and tender wheels are taken together in determining the center of gravity. As the weight carried on the truck is largely a matter of adjustment, it seems wise to eliminate the truck wheels in a typical engine and to add their weight to

the driving wheels. If these changes are made and the total weight of the engine is increased to 300 000 lbs., or 5 000 lbs. per foot, the result is the typical engine shown in Fig. 3. This engine does not differ materially from that shown in Fig. 4 in any of its effects on a bridge. Mr. Morison.

Fig. 3 is an exceedingly simple engine so far as its weight on the tracks is concerned. On each of the four driving axles is a weight of 50 000 lbs. On each of the four tender axles is a weight of 25 000 lbs., or just half that of the driving axles; the center of gravity of one set of wheels is just 30 ft. from the center of gravity of the other set of wheels, and every distance is a multiple of 5.

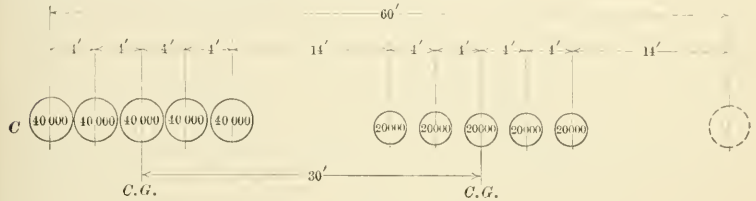


Fig. 4.

Now, this engine does not differ materially, in any of its effects on a bridge, from the engine shown in Fig. 4, which is a ten-wheel coupled locomotive with a tender also having ten wheels. Fig. 4 is a peculiar engine, which has been somewhat used, but it is put in here simply because, though the nominal wheel base is a little more, the actual wheel base is practically the same, and in the effects of the two engines on a bridge hardly any difference can be found.

These engines do not represent any now running, but are typical engines for use in calculation, and producing more excessive effects

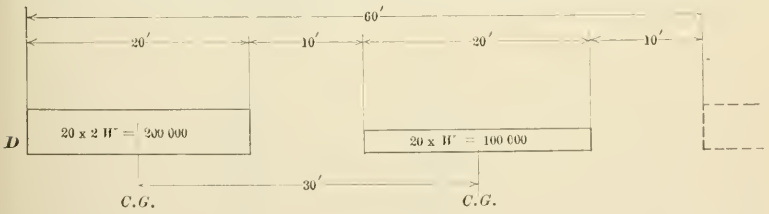


Fig. 5.

than engines of the same weight built on any of the standard lines. Furthermore, they are engines of such form that they can be used very easily in all calculations.

It will be observed that the weight on the driving wheels is exactly double that on the other wheels, that the two sets of wheels are arranged in precisely the same way around their centers of gravity, and that the distance between the two centers of gravity, is half the length of the locomotive, or 30 ft.

If we substitute for the specially applied wheel load, a uniform

Mr. Morison. load of equal intensity, of a length equal to the number of wheels multiplied by the distance between their centers, we have the results shown in Fig. 5.

This loading will also represent passenger locomotives of rather extraordinary dimensions. For instance, the engine wheel-base load of 200 000 may be considered as representing a locomotive with four driving wheels and 50 000 lbs. on each driving wheel; while it is to be hoped that no such locomotive will ever be built, pressures of approximately 50 000 lbs. per wheel now exist in badly counterbalanced engines when running at excessive speeds. That is a thing which bridge specifications have never taken into consideration, and which throws all the ordinary refinements of wheel-base loads out of gear. These strains, however, do not exist on all four wheels at the same time; their influence is felt on single stringers of short lengths, but not on complete spans.

In order to show how closely this typical loading agrees with the several locomotives, Fig. 6 has been prepared, and shows the bending moments under the several conditions. This drawing is taken on the basin of a 60-ft. span with the center of the driving-wheel base placed at the center of the span. The moments of the five different engines and loadings which we have been considering are drawn to scale.

Taking a moving load composed of a continuous train of the typical "*D*" loading (Fig. 5) as a basis of calculation, Table No. 1 shows the maximum shearing strains at the end of the several spans named under such moving load, the lengths of span varying by 20 ft.

TABLE No. 1.

Length of span. Feet.	*a* End shear by typical load.	*b* End shear by uniform load (5 000 lbs. per linear foot).	$a \div b$ Coefficients.
20	100 000	50 000	2.000
40	156 250	100 000	1.563
60	200 000	150 000	1.333
80	250 000	200 000	1.250
100	302 500	250 000	1.210
120	350 000	300 000	1.167
140	400 000	350 000	1.143
160	451 563	400 000	1.129
180	500 000	450 000	1.111
200	550 000	500 000	1.100
220	601 136	550 000	1.093
240	650 000	600 000	1.083
260	700 000	650 000	1.077
280	750 893	700 000	1.073
300	800 000	750 000	1.067
320	850 000	800 000	1.063
340	900 735	850 000	1.059
360	950 000	900 000	1.056
380	1 000 000	950 000	1.053
400	1 050 625	1 000 000	1.051
420	1 100 000	1 050 000	1.048
440	1 150 000	1 100 000	1.045
460	1 200 543	1 150 000	1.044
480	1 250 000	1 200 000	1.042
500	1 300 000	1 250 000	1.040

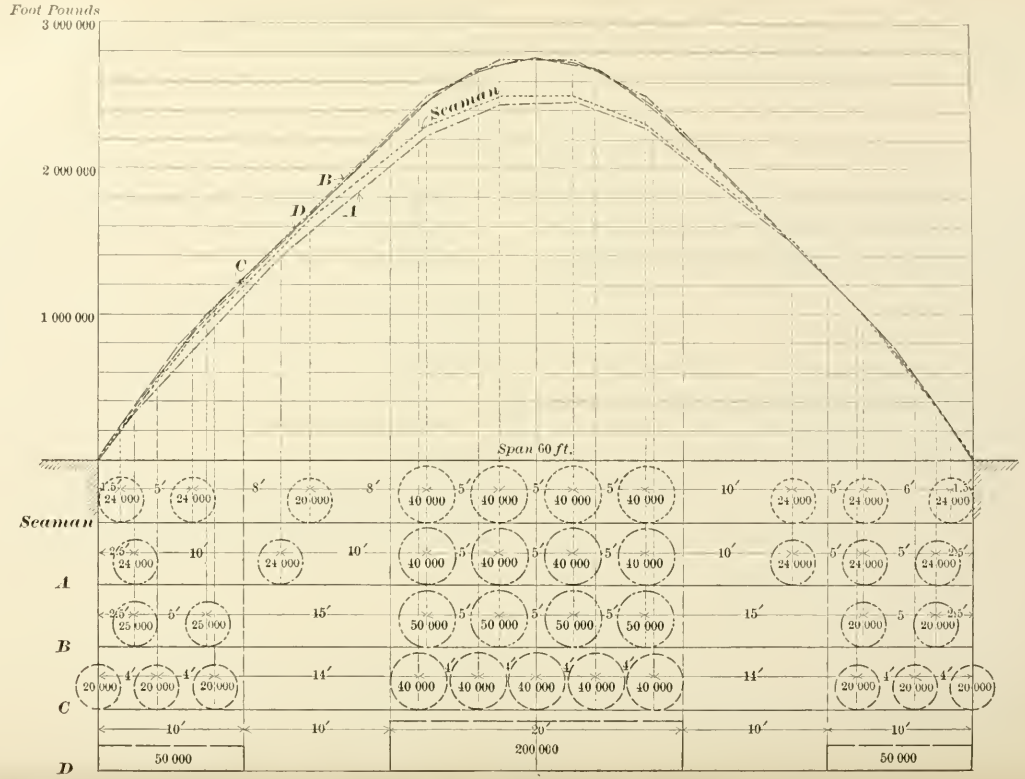

Foot Pounds
3 000 000
2 000 000
1 000 000
Seaman
B
D
A
C
Span 60 ft.
Seaman
A
B
C
D
50 000
200 000
50 000

FIG. 6.

Mr. Morison. In the same way the bending moments in the center of the span are given in Table No. 2.

TABLE No. 2.

Length of Span. Feet.	a Moment by typical load.	b Moment by uniform load (5 000 lbs. per linear foot).	$a \div b$ Coefficients.
20	500 000	250 000	2.000
40	1 500 000	1 000 000	1.500
60	2 750 000	2 250 000	1.222
80	4 500 000	4 000 000	1.125
100	6 500 000	6 250 000	1.040
120	9 000 000	9 000 000	1.000
140	12 500 000	12 250 000	1.025
160	16 500 000	16 000 000	1.031
180	20 750 000	20 250 000	1.025
200	25 500 000	25 000 000	1.020
220	30 500 000	30 250 000	1.008
240	36 000 000	36 000 000	1.000
260	42 500 000	42 250 000	1.006
280	49 500 000	49 000 000	1.010
300	56 750 000	56 250 000	1.009
320	64 500 000	64 000 000	1.008
340	72 500 000	72 250 000	1.003
360	81 000 000	81 000 000	1.000
380	90 500 000	90 250 000	1.003
400	100 500 000	100 000 000	1.005
420	110 750 000	110 250 000	1.0045
440	121 500 000	121 000 000	1.004
460	132 500 000	132 250 000	1.002
480	144 000 000	144 000 000	1.000
500	156 500 000	156 250 000	1.002

These calculations are based on a train consisting entirely of locomotives, and such trains are not run. Approximation to such trains are found, however, in mineral traffic, in cars of special character. A car designed and built by the Goodwin Patent Gravity Dumping Car Company weighs 42 000 lbs., and is designed to carry 125 000 lbs. of ore, making a total weight of 167 000 lbs. on a length a trifle less than 36 ft., or very nearly 5 000 lbs. per foot. In mineral districts it would seem expedient to proportion bridges for weights equivalent to trains composed entirely of locomotives. In other districts it may be expedient to provide for only two locomotives and to reduce the weight of the following train to something less than the locomotive load. If this reduction is arbitrarily taken at 25%, the train load per foot will be three-quarters of the locomotive load per foot. It seems right to apply this correction in the form of a reduction from a uniform load rather than to make the locomotive an addition to it. On this basis Tables Nos. 3 and 4 have been calculated, Table No. 3 showing the maximum shearing strain at the ends and Table No. 4 the maximum bending moments at the centers of spans, caused by a train consisting of two locomotives followed by a uniform load of three-quarters of the average weight of the locomotives. It will be observed that these two tables are identical with the foregoing tables for all lengths under 120 ft.

Mr. Morison.

TABLE No. 3.

Length of span. Feet.	*a* End shear by typical load.	*b* End shear by uniform load (5 000 lbs. per lineal foot).	$a \div b$ Coefficients.
20	100 000	50 000	2.000
40	156 250	100 000	1.563
60	200 000	150 000	1.333
80	250 000	200 000	1.250
100	302 500	250 000	1.210
120	350 390	300 000	1.168
140	394 075	350 000	1.126
160	436 230	400 000	1.090
180	477 345	450 000	1.061
200	517 734	500 000	1.035
220	557 590	550 000	1.014
240	596 860	600 000	0.994
260	636 238	650 000	0.979
280	675 165	700 000	0.964
300	713 905	750 000	0.952
320	752 500	800 000	0.941
340	791 000	850 000	0.931
360	829 400	900 000	0.921
380	867 555	950 000	0.913
400	905 740	1 000 000	0.906
420	943 860	1 050 000	0.899
440	981 925	1 100 000	0.893
460	1 019 940	1 150 000	0.889
480	1 057 910	1 200 000	0.882
500	1 095 840	1 250 000	0.877

TABLE No. 4.

Length of span. Feet.	*a* Moment by typical load.	*b* Moment by uniform load (5 000 lbs. per lineal foot).	$a \div b$ Coefficients.
20	500 000	250 000	2.000
40	1 500 000	1 000 000	1.500
60	2 750 000	2 250 000	1.222
80	4 500 000	4 000 000	1.125
100	6 523 437	6 250 000	1.044
120	9 037 500	9 000 000	1.004
140	12 260 187	12 250 000	1.001
160	15 718 750	16 000 000	0.982
180	19 413 437	20 250 000	0.959
200	23 336 250	25 000 000	0.933
220	27 490 234	30 250 000	0.909
240	31 875 000	36 000 000	0.885
260	36 240 230	42 250 000	0.858
280	41 583 435	49 000 000	0.849
300	46 837 500	56 250 000	0.833
320	52 500 000	64 000 000	0.820
340	58 687 500	72 250 000	0.812
360	65 250 000	81 000 000	0.805
380	72 187 500	90 250 000	0.800
400	79 500 000	100 000 000	0.795
420	87 185 500	110 250 000	0.791
440	95 250 000	121 000 000	0.787
460	103 687 500	132 250 000	0.784
480	112 500 000	144 000 000	0.781
500	121 687 500	156 250 000	0.779

The coefficients of the four tables have been plotted on Fig. 7.

Mr. Morison.

If these coefficients have to be calculated for every position of loading, there is no advantage in their use; furthermore, the coefficients of Table No. 2 show a series of maxima and minima which ought to be eliminated. To obviate these difficulties a few simple rules are proposed for fixing coefficients for every length of span; the coefficients, as determined by these rules will, in almost all cases, be slightly in excess of the calculated coefficients. The coefficients, as determined by these rules, are plotted as full lines in Fig. 7.

Rule 1.—Bending Moments. Uniform Locomotive Load.—Coefficient for 220 ft., and all greater spans, 1. Increase this coefficient by one-tenth of 1% (0.001) for each foot of decrease of length down to 120 ft., making the coefficient for 120 ft. 1.10. Increase this by one-third of 1% (0.0033) for each ft. down to 60 ft., making the coefficient for 60 ft. 1.3. Increase by 1.75% (0 0175) for each foot down to 20 ft., making the coefficient for 20 ft. 2.

Rule 2.—Bending Moments. Two Locomotives Followed by Train.—Coefficient for 360 ft. and all greater spans, 0.8. Increase this coefficient by one-eighth of 1% (0.00125) for each foot of decrease of length down to 120 ft., making the coefficient for 120 ft. 1.1. Increase by one-third of 1% (0.0033), for each foot down to 60 ft., making the coefficient for 60 ft. 1.3. Increase by 1.75% (0.0175), for each foot down to 20 ft., making the coefficient for 20 ft. 2.

Rule 3.—Shearing Strains. Uniform Locomotive Load.—Coefficient for 420 ft. and all greater spans, 1.05. Increase this coefficient by one-fortieth of 1% (0.00025), for each foot of decrease of length down to 220 ft., making the coefficient for 220 ft. 1.1. Increase by one-tenth of 1% (0.001), for each foot down to 120 ft., making the coefficient for 120 ft. 1.2. Increase by one-sixth of 1% (0.00167), for each foot down to 60 ft., making the coefficient for 60 ft. 1.3. Increase by 1.75% (0.0175), for each foot down to 20 ft., making the coefficient for 20 ft. 2.

Rule 4.—Shearing Strains. Two Locomotives in Train Load.—Coefficient for 360 ft. and all greater spans, 0.9. Increase this coefficient by one-eighth of 1% (0.00125), for each foot of decrease of length down to 120 ft., making the coefficient for 120 ft., 1.2. Increase by one-sixth of 1% (0.00167), for each foot down to 60 ft., making the coefficient for 60 ft., 1.3. Increase by 1.75% (0.0175), for each foot down to 20 ft., making the coefficient for 20 ft., 2.

These rules are designed principally as a means of carrying easily and in compact shape these coefficients so that they can be handled quickly, without tabulation. A better method, however, would be to draw a curve governed by the different points which are used in the calculations, and then determine the coefficient for each foot of length from that curve. The speaker has endeavored to find some curve, with a simple formula, which might be used in this way, but, so far, he has not succeeded. Perhaps some other member of the Society may be more fortunate.

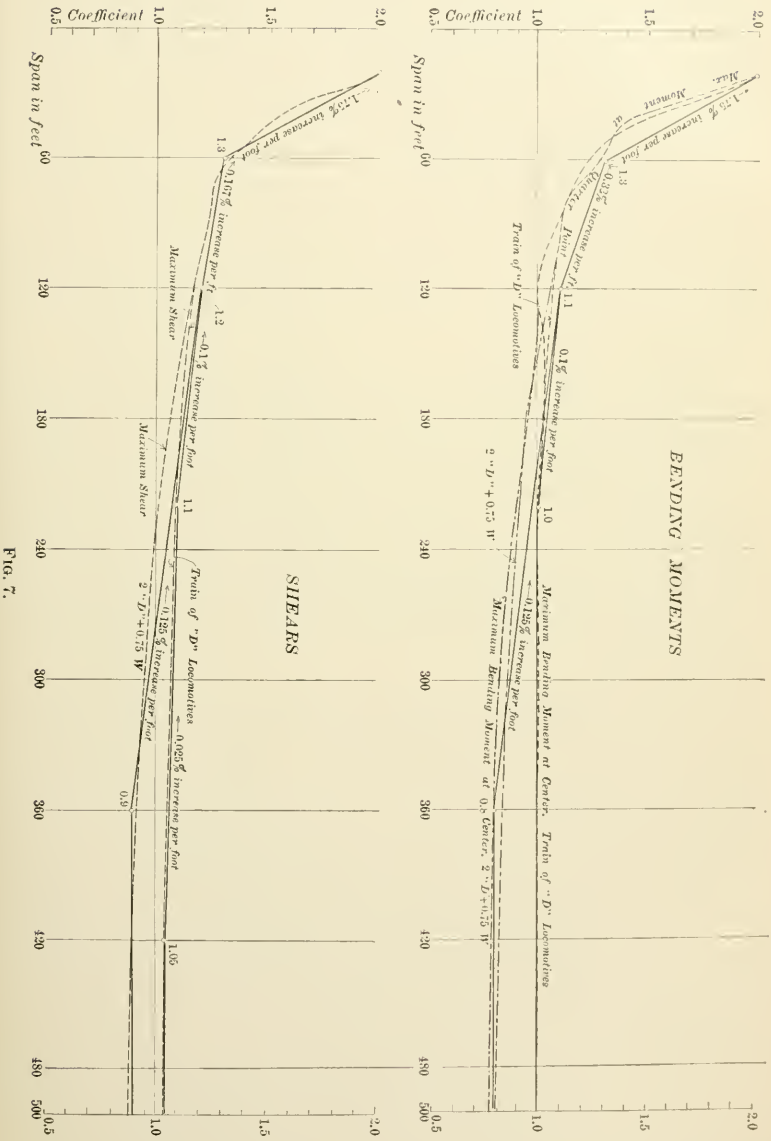

Coefficient
Span in Feet
BENDING MOMENTS
Maximum Bending Moment at Center. Train of "D" Locomotives
Maximum Bending Moment at 0.5 Center. 2 "D" + 0.75 W
Train of "D" Locomotives
2 "D" + 0.75 W
0.125% increase per foot
0.1% increase per foot
1.3% increase per foot
Moment at
Maximum
Theoretical
Points
SHEARS
Maximum Shear
Train of "D" Locomotives
2 "D" + 0.75 W
0.125% increase per foot
0.025% increase per foot
0.167% increase per ft.
0.1% increase per foot
1.3% increase per foot

Fig. 7.

Mr. Cowles. W. L. Cowles, M. Am. Soc. C. E.—The admirable paper entitled "General Criterion for Position of Loads causing Maximum Stresses in any Member of a Bridge Truss," presented by L. M. Hoskins, Esq., affords an interesting mathematical treatment of the problem, and is a valuable contribution to the theoretical literature on this subject.

The speaker desires, however, at the present time, to discuss briefly the necessity for finding this position, rather than the method of accomplishing the object.

Such a discussion naturally merges with the discussion of concentrated *versus* uniform loads, for the main object to be finally attained is the determination of the maximum stresses produced in the members of a truss by the application of a load within a satisfactory degree of accuracy and with the least labor on the part of the computer.

It must be conceded that a bridge is rarely loaded with the exact load for which it is designed, and in fixing upon the load to be used in making the calculations it is the object of the engineer to specify such a load, either uniform or composed of concentrated wheel loads, as will produce stresses at least equal to, and preferably somewhat greater than will ever be reached in common practice, making due allowance for any probable increase in loading in the future. This being the case, it follows that extreme accuracy in the calculations is not essential, and, given a satisfactory degree thereof, the problem reduces to a determination of the simplest means of arriving at the results sought.

For ease and quickness of computation it is beyond doubt that a uniform load is preferable, and the speaker is in favor of the adoption of such loads, if the railroad engineers can be induced to specify them when requesting propositions for bridge work. Notwithstanding, however, the extended discussion of this subject by the members of this Society, and the offers which have been made of tables indicating equivalent loads, a comparison of the bridge specifications of the railroads of the United States proves that, with a very few exceptions, the engineers of these roads still prefer to specify a loading consisting of a series of concentrated loads representing typical locomotives followed by a uniform train load, and, in the computing department of a bridge works, it becomes necessary to take things as they are rather than as they might be.

It becomes, then, an important matter to determine the simplest manner of preparing the stress sheet and especially within what limits it is necessary to determine the special position of the load which will produce in each member the maximum stress. With this object in view, the speaker has prepared comparative stress sheets for spans of 125, 150, 175, 200 and 300 ft., based upon loading "$E40$" of Cooper's

Specifications, the stresses being computed by five different methods, and has tabulated the results showing the differences arising from the use of the respective methods (Table No. 5). The basis of the comparison is Method *A*, in which the position of the load is shifted, in accordance with the formulas of the author, so as to produce the maximum stress in each member. Method *B* is based upon the use of a fixed series of panel loads headed by the maximum floor-beam concentration, progressing on the bridge a panel at a time for successive shears, and covering the entire bridge for all chord moments. Mr. Cowles.

The effect of the small panel load in advance of the maximum head load is disregarded. Method *C* is based upon a uniform load which will produce the same end shear as Method *A*. Method *D* is based upon a uniform load which will produce the same moment at the center as Method *A*. Method *E* is based upon a uniform load equal to the following train load with two additional concentrated loads, two panel lengths apart, which will produce the same end shear as Method *A*. The quantities given are in thousands of pounds, and represent shears not multiplied by the secant in the case of web members, and chord stresses for a depth equal to a panel length, that is, not multiplied by the tangent. A uniform panel length of 25 ft. was adopted.

The allowable limit of variance from the strictly correct stresses which may be permitted for the sake of simplifying the computation is two-fold: First, the percentage of deficiency in any member must not exceed a certain amount; second, the percentage of excess in the total weight of the structure must not exceed a certain amount, and that assumption, for an equivalent load which will result in the smallest percentages of variance and at the same time afford a simple means of preparing the stress sheet, is the one which will naturally be selected. An inspection of the table will show clearly the limit of variance involved in each of these methods, but in general it may be stated that Method *B* gives the smallest percentages, taking both considerations into account. That is, while Method *C* avoids deficient stresses in the most important members, it requires a larger excess of total material; while Method *D* results in a reduction of total weight, the deficiencies in stresses are large; and while Method *E* avoids all deficiencies in stresses, the excess of total weight is even greater than for Method *C*. The maximum deficiency in stress resulting from the use of Method *B* is, for a 150-ft. span, nothing, for a 175-ft. span 0.1 of 1%, for a 200-ft. span 2.1% in one member only, for a 300-ft. span 1.8%; while the excess of total weight is, for the same spans, respectively, 0.8, 0.7, 0.4 of 1% and less than 0.1 of 1 per cent.

The permissible limit of variance is a matter which each engineer will fix for himself in accordance with his own sense of propriety, but it seems to the speaker that the percentages mentioned for Method *B*

Mr. Cowles. are not excessive, when it is considered that the assumed load itself should be greater than the probable actual load by a margin at least as great as the largest of these percentages. It will be remembered also that, in the case of a new specification coming into a computer's hands, the work involved in Method *B* or its equivalent, must first be done in order to determine the uniform load which may be used in Methods *C*, *D* or *E*, and it is to be noted that still another uniform load must be determined for the floor-beam moment and the stress on the hip vertical. It may be stated here that in determining the percentage of excess weight for Methods *C* and *D* it was assumed that the stress on the hip vertical given in the table would not be used, but that this member would be brought up to proper size as just suggested. The calculation for stringers would, of course, be the same for each method.

If the engine excess for Method *E* be determined by taking the difference between the actual engine weight and the weight of a section of the following load equal in length to the length of the engine, the resulting stresses and the excess weight will be even greater than given in the table.

In case a certain specification is likely to be used frequently, and a method of uniform load is preferred, it is naturally advantageous to prepare a series of uniform loads, corresponding to different lengths of spans, for use in computations; and it cannot be contended that this will not effect a saving in time after the series is once prepared, but the amount of this saving is, in the opinion of the speaker, much overestimated in discussing the relative merits of the different methods. An important factor affecting the simplicity of a computation is the orderly and systematic arrangement of the mathematical work, and in this matter each engineer will adopt such a system as to him will seem the quickest and easiest to obtain the desired result. The speaker has been accustomed to using Method *B*, and the system adopted by him, which is doubtless used by others, affords a reasonably quick process and one which compares favorably with the method of uniform loads. He recently noted the time occupied in making the computations for the truss members of a 300-ft. span with horizontal top chord and having 13 panels, giving odd panel lengths. The load used was "Cooper's *E* 40." Having first determined the panel length, which must be done in any case, the panel loads were determined in 17 minutes and the complete shears and moments, ready for multiplying by secant and tangent, in 18 minutes more, or 35 minutes in all.

There does not seem to be much room here for saving in time by the use of another method, even if the equivalent uniform or excess loads have already been determined.

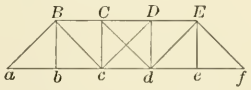
There is nothing new claimed for this method; in fact it may be

called old-fashioned, but this comparison indicates that it gives reasonably close results and is not laborious or slow. Mr. Cowles.

The speaker has not had time to make similar comparisons for subdivided or curved top trusses, but has no reason to suspect that the limits of variance would be any greater in these cases than in those investigated, and he therefore feels confident that, while railroad engineers retain a preference for a type of loading which they can readily compare with actual engines in use on their lines, satisfactory stress sheets may be prepared without undue expenditure of time and labor. He will, however, welcome such a change of attitude, on the part of these engineers, as will result in a reduction in the number of varying loadings and admit of the adoption of a limited number of standard loads, preferably uniform.

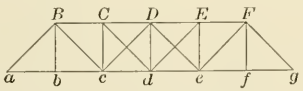
TABLE No. 5.

125-FT. SPAN.



Member.	Method A.	Method B.	Diff.	Per cent.	Method C.	Diff.	Per cent.	Method D.	Diff.	Per cent.	Method E.	Diff.	Per cent.
B C....	194.6	190.0	—4.6	2.4	199.2	+4.6	.....	194.6	.....	.....	193.7	—0.9	0.5
a c.....	132.8	132.8	.....	.....	132.8	.....	.....	129.7	—3.1	2.3	132.8	.....	.....
c d.....	190.0	190.0	.....	.....	199.2	+9.2	.....	194.6	+4.6	.....	193.7	+3.7	.....
B b....	75.6	75.6	.....	.....	66.4	—9.2	12.2	64.9	—10.7	14.2	77.3	+1.7	.....
C c.....	36.4	40.8	+4.4	.....	39.8	+3.4	.....	38.9	+2.5	.....	40.9	+4.5	.....
B a....	132.8	132.8	.....	.....	132.8	.....	.....	129.7	—3.1	2.3	132.8	.....	.....
B c.....	77.3	81.0	+3.3	.....	79.7	+2.4	.....	77.8	+0.5	.....	81.9	+4.6	.....
C d.....	36.4	40.8	+4.4	.....	39.8	+3.4	.....	38.9	+2.5	.....	40.9	+4.5	.....
Excess weight..			.....	+0.5		.....	+1.1		.....	0.0		.....	+1.0

150-FT. SPAN.

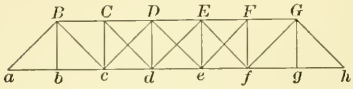


Member.	Method A.	Method B.	Diff.	Per cent.	Method C.	Diff.	Per cent.	Method D.	Diff.	Per cent.	Method E.	Diff.	Per cent.
B C....	248.3	248.3	.....	.....	259.2	+10.1	.....	250.8	+2.5	.....	255.5	+7.2	.....
C D....	282.1	282.1	.....	.....	291.6	+ 9.5	.....	282.1	.....	.....	280.5	—1.6	0.6
a c.....	162.0	162.0	.....	.....	162.0	.....	.....	156.7	— 5.3	3.3	162.0	.....	.....
c d.....	248.3	248.3	.....	.....	259.2	+10.1	.....	250.8	+ 2.5	.....	255.5	+7.2	.....
B b.....	75.6	75.6	.....	.....	64.8	—10.8	14.3	62.7	—12.9	17.1	77.8	+2.2	.....
C c.....	62.9	67.5	+4.6	.....	64.8	+ 1.9	.....	62.7	— 0.2	0.3	68.5	+5.6	.....
D d.....	29.7	34.0	+4.3	.....	32.4	+ 2.7	.....	31.4	+ 1.7	.....	34.3	+4.6	.....
B a.....	162.0	162.0	.....	.....	162.0	.....	.....	156.7	— 5.3	3.3	162.0	.....	.....
B c.....	107.6	110.7	+3.1	.....	108.0	+ 0.4	.....	104.5	— 3.1	2.9	111.1	+3.5	.....
C d.....	62.9	67.5	+4.6	.....	64.8	+ 1.9	.....	62.7	— 0.2	0.3	68.5	+5.6	.....
D e.....	29.7	34.0	+4.3	.....	32.4	+ 2.7	.....	31.4	+ 1.7	.....	34.3	+4.6	.....
Excess weight..			.....	+0.8		.....	+1.1		.....	—0.5		.....	+1.2

Mr. Cowles.

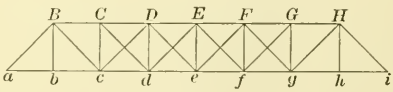
TABLE No. 5—(*Continued*).

175-FT. SPAN.



Member.	Method A.	Method B.	Diff.	Per cent.	Method C.	Diff.	Per cent.	Method D.	Diff.	Per cent.	Method E.	Diff.	Per cent.
B C....	304.8	304.3	−0.5	0.1	316.7	+11.9	.....	305.0	+ 0.2	.....	314.0	+ 9.2	.....
C D....	366.0	366.0	.....	.....	380.0	+14.0	.....	366.0	.....	.....	377.0	+11.0	.....
a c....	190.0	190.0	.....	.....	190.0	.....	.....	183.0	− 7.0	3.7	190.0	.....	.....
c d....	304.8	304.3	−0.5	0.1	316.7	+11.9	.....	305.0	+ 0.2	.....	314.0	+ 9.2	.....
d e....	363.3	366.0	+2.7	.....	380.0	+16.7	.....	366.0	+ 2.7	.....	377.0	+13.7	.....
B b....	75.6	75.6	.....	.....	63.3	−12.3	16.3	61.0	−14.6	19.3	78.0	+ 2.4	.....
C c....	90.9	94.9	+4.0	.....	90.5	− 0.4	0.4	87.1	− 3.8	4.2	95.4	+ 4.5	.....
D d....	52.6	57.9	+5.3	.....	54.3	+ 1.7	.....	52.3	− 0.3	0.6	58.8	+ 6.2	.....
B a....	190.0	190.0	.....	.....	190.0	.....	.....	183.0	− 7.0	3.7	190.0	.....	.....
B c....	136.2	138.9	+2.7	.....	135.7	− 0.5	0.4	130.7	− 5.5	4.0	139.1	+ 2.9	.....
C d....	90.9	94.9	+4.0	.....	90.5	− 0.4	0.4	87.1	− 3.8	4.2	95.4	+ 4.5	.....
D e....	52.6	57.9	+5.3	.....	54.3	+ 1.7	.....	52.3	− 0.3	0.6	58.8	+ 6.2	.....
E f....	25.0	29.1	+4.1	.....	27.2	+ 2.2	.....	26.2	+ 1.2	.....	29.4	+ 4.4	.....
Excess weight...			.....	+0.7	.....	.....	+0.9	.....	.....	.....	.....	.....	+1.4

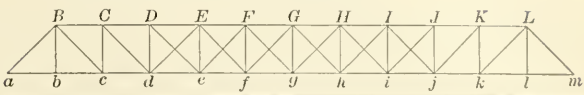
200-FT. SPAN.



Member.	Method A.	Method B.	Diff.	Per cent.	Method C.	Diff.	Per cent.	Method D.	Diff.	Per cent.	Method E.	Diff.	Per cent.
B C....	359.8	358.7	− 1.1	0.3	372.4	+12.6	.....	355.4	− 4.4	1.2	370.3	+11.1	.....
C D....	448.5	447.6	− 0.9	0.2	465.5	+17.0	.....	444.2	− 4.3	1.0	459.3	+10.9	.....
D E....	473.8	463.8	−10.0	2.1	496.5	+22.7	.....	473.8	.....	.....	484.3	+10.5	.....
a c....	217.2	217.2	.....	.....	217.2	.....	.....	208.1	− 9.1	4.2	217.2	.....	.....
c d....	359.8	358.7	− 1.1	0.3	372.4	+12.6	.....	355.4	− 4.4	1.2	370.3	+11.1	.....
d e....	448.5	447.6	− 0.9	0.2	465.5	+17.0	.....	444.2	− 4.3	1.0	459.3	+10.9	.....
B b....	75.6	75.6	.....	.....	62.1	−13.7	18.1	59.2	−16.4	21.7	78.1	+ 2.5	.....
C c....	118.0	121.5	+ 3.5	.....	116.4	− 1.6	1.4	111.0	− 7.0	5.9	121.9	+ 3.9	.....
D d....	78.4	83.0	+ 4.6	.....	77.6	− 0.8	1.0	74.0	− 4.4	5.6	83.6	+ 5.2	.....
E e....	45.1	50.6	+ 5.5	.....	46.6	+ 1.5	.....	44.6	− 0.5	1.1	51.5	+ 6.4	.....
B a....	217.2	217.2	.....	.....	217.2	.....	.....	208.1	− 9.1	4.2	217.2	.....	.....
B c....	163.9	166.2	+ 2.3	.....	162.9	− 1.0	0.6	156.1	− 7.8	4.8	166.4	+ 2.5	.....
C d....	118.0	121.5	+ 3.5	.....	116.4	− 1.6	1.4	111.0	− 7.0	5.9	121.9	+ 3.9	.....
D e....	78.4	83.0	+ 4.6	.....	77.6	− 0.8	1.0	74.0	− 4.4	5.6	83.6	+ 5.2	.....
E f....	45.1	50.6	+ 5.5	.....	46.6	+ 1.5	.....	44.6	− 0.5	1.1	51.5	+ 6.4	.....
F g....	21.5	25.5	+ 4.0	.....	23.3	+ 1.8	.....	22.3	+ 0.8	3.7	25.8	+ 4.3	.....
Excess weight..			.....	+ 0.4	.....	.....	+ 1.0	.....	.....	.....	.....	.....	+1.4

TABLE No. 5—(*Concluded*). Mr. Cowles.

300-Ft. Span.



Member.	Method A.	Method B.	Diff.	Per cent.	Method C.	Diff.	Per cent.	Method D.	Diff.	Per cent.	Method E.	Diff.	Per cent.
B C....	571.2	569.3	— 1.9	0.3	586.4	+15.2	.....	542.3	—28.9	5.1	585.5	+14.3	.....
C D....	765.1	763.5	— 1.6	0.2	791.6	+26.5	.....	732.1	—33.0	4.3	789.0	+23.9	.....
D E....	899.2	885.0	—14.2	1.6	938.2	+39.0	.....	867.7	—31.5	3.5	933.0	+33.8	.....
E F....	965.3	948.2	—17.1	1.8	1026.2	+60.9	.....	949.0	—16.3	1.7	1017.5	+52.2	.....
F G....	976.2	962.9	—13.3	1.4	1055.5	+79.3	.....	976.2	.....	.....	1042.5	+66.3	.....
a c.....	322.5	322.5	.....	.....	322.5	.....	.....	298.3	—24.2	7.5	322.5	.....	.....
c d.....	571.2	569.3	— 1.9	0.3	586.4	+15.2	.....	542.3	—28.9	5.1	585.5	+14.3	.....
d e.....	765.1	763.5	— 1.6	0.2	791.6	+26.5	.....	732.1	—33.0	4.3	789.0	+23.9	.....
e f.....	899.2	885.0	—14.2	1.6	938.2	+39.0	.....	867.7	—31.5	3.5	933.0	+33.8	.....
f g.....	965.3	948.2	—17.1	1.8	1026.2	+60.9	.....	949.0	—16.3	1.7	1017.5	+52.2	.....
B b.....	75.6	75.6	.....	.....	58.6	—17.0	22.5	54.2	—21.4	28.3	78.5	+ 2.9	.....
C c.....	223.0	225.3	+ 2.3	.....	220.0	— 3.0	1.4	203.4	—19.6	8.8	225.5	+ 2.5	.....
D d.....	179.9	183.0	+ 3.1	.....	175.9	— 4.0	2.2	162.7	—17.2	9.6	183.3	+ 3.4	.....
E e.....	140.9	144.8	+ 3.9	.....	136.8	— 4.1	2.9	126.6	—14.3	10.1	145.2	+ 4.3	.....
F f.....	106.2	110.8	+ 4.6	.....	102.6	— 3.6	3.4	94.9	—11.3	10.6	111.3	+ 5.1	.....
G g.....	76.0	81.0	+ 5.0	.....	73.3	— 2.7	3.6	67.8	— 8.2	10.8	81.5	+ 5.5	.....
B a.....	322.5	322.5	.....	.....	322.5	.....	.....	298.3	—24.2	7.5	322.5	.....	.....
B c.....	270.2	271.8	+ 1.6	.....	268.7	— 1.5	0.6	248.6	—21.6	8.0	271.9	+ 1.7	.....
C d.....	223.0	225.3	+ 2.3	.....	220.0	— 3.0	1.4	203.4	—19.6	8.8	225.5	+ 2.5	.....
D e.....	179.9	183.0	+ 3.1	.....	175.9	— 4.0	2.2	162.7	—17.2	9.6	183.3	+ 3.4	.....
E f.....	140.9	144.8	+ 3.9	.....	136.8	— 4.1	2.9	126.6	—14.3	10.1	145.2	+ 4.3	.....
F g.....	106.2	110.8	+ 4.6	.....	102.6	— 3.6	3.4	94.9	—11.3	10.6	111.3	+ 5.1	.....
G h.....	76.0	81.0	+ 5.0	.....	73.3	— 2.7	3.6	67.8	— 8.2	10.8	81.5	+ 5.5	.....
H i.....	50.4	55.3	+ 4.9	.....	48.9	— 1.5	3.0	45.2	— 5.2	10.3	55.9	+ 5.5	.....
I j.....	29.0	33.7	+ 4.7	.....	29.3	+ 0.3	.....	27.1	— 1.9	6.6	34.5	+ 5.5	.....
Excess weight..			.....	0.0	.....	.....	+ 1.2	.....	.....	.....	.....	.....	+1.7

E. M. Scofield, Assoc. M. Am. Soc. C. E.—For several years the speaker's time has been devoted to the designing of truss bridges. He has used both methods of calculation, and is not in favor of the method of wheel loads. For each new engine loading it is necessary to make a moment table, which has to be checked carefully, and, up to this point, the two methods are the same. Aside from that, the speaker can compute uniform loads from end shear and figure the stresses in a bridge in about one-quarter to one-tenth of the time required by the exact wheel-load method, thus saving much time and largely avoiding danger from mistakes. Mr. Scofield.

By the uniform method, after checking the end shear, all the stresses in the bridge can be found in from 15 to 20 minutes, and checked beyond all possibility of error. By the wheel-load method the speaker has never been able to make the calculations and check them in such short time; it generally takes from four to ten times as long, and then does not afford such a ready means of checking. In fact, the speaker has seldom made such calculations by wheel loads without making in parallel columns similar calculations by uniform load as a check; and he finds that such a method of checking is more reliable than to attempt to check a wheel-load calculation.

Mr. Scofield. In using the wheel-load method of calculation, the number of manipulations is so great that a man who is figuring with various engines and various specifications is in great danger of making mistakes; whereas the simplicity of the equivalent uniform-load method is such that the calculations can be checked in a very short time.

While there are many different methods of determining equivalent uniform loads, that which bases such loads on the shear in the end panel, is, in the opinion of the speaker, most satisfactory for truss calculations.

All engineers do not agree with this view, but a uniform load based on the shear in the end panel will give stresses which are exact for the end post and nearly exact for the first tie, the error being always on the side of safety and increasing slowly toward the center and being greatest in the counters where the tonnage involved is small and where an excess of metal is to be commended. This method also gives in the chords a little too much sectional area, but, judging from actual tests of bottom chords, under everyday conditions, a little excess of metal in those members is likely to be well placed.

The argument for exact wheel-loads is altogether too refined, because the engine specified may never pass over the bridge, and the variation due to the use of a uniform load will not exceed 4% in ordinary cases and will not be nearly so much in most of the members where the weight is concentrated.

In the opinion of the speaker, it is well that wheel loads be specified simply because they are more intelligible, especially to the buyer. A railroad engineer who is not dealing with bridges all the time, or the engineer whose time is mostly devoted to other kinds of work, will prefer to have wheel loads specified because they mean something definite to him, which the uniform load does not.

The speaker believes that, in addition to specifying a typical engine or an actual engine, with its following load, a uniform load obtained in a certain, definite way should also be specified and required to be used in making the calculations. The speaker would specify loads based on the center moment for stringers and floor beams and on the moment at the quarter point for girders, and on the shear in the end panel for trusses. This method would give a better bridge and would save time in making the calculations, which would, in the end, be a saving to the buyer. There are other methods for obtaining equivalent loads which are not without merit and are alike in being preferable to the wheel-load method. It would be an additional advantage if the equivalent loads for spans varying by small intervals were given.

Mr. Seaman. Henry B. Seaman, M. Am. Soc. C. E.—Any movement toward the adoption of a train load with a single concentration, instead of what is termed a "typical" engine, but which has really become an equivalent typical engine, should be encouraged. About fifteen years ago it

Mr. Seaman.

was the custom to specify all the various types of engines which might in any way produce the maximum strain under varying conditions. Calling to mind especially the Pennsylvania Railroad Specifications, which were presented to the Society about that time by Joseph M. Wilson, M. Am. Soc. C. E., three typical engines were used. These were truly typical, and represented very closely the actual weights and wheel spacing of the various types of engines in use, such as the consolidation, the passenger and the switch engine. But as general specifications were written for adoption upon various roads, it was found expedient to use one engine diagram which would produce strains approximately equivalent to the maximum from the three types. A consolidation engine diagram was outlined for this purpose, and this might properly be termed an equivalent typical engine, *i. e.*, producing strains approximately equivalent to the three types formerly used.

Some ten years ago the speaker had occasion to investigate the weights of engines for C. W. Buchholz, M. Am. Soc. C. E., Chief Engineer of the Erie Railroad, and the result of this study was given in a discussion of the paper on "Some Disputed Points in Railway Bridge Designing." There was no engine then in use with a weight on the pony wheel exceeding 16 000 lbs.; and the weights on an axle of the tender in no case would exceed 23 000 lbs., with full weight of water, and with sides boarded up to carry an extra heavy load of coal. To-day the typical engines, or rather the equivalent typical engines, are shown with 20 000 lbs. on the pony wheel, and in some instances with 26 000 lbs. on each tender axle, which is a variation of 10 to 25% from actual conditions. In other words, we are designing bridges for loads which do not exist, and which cannot come upon them.

Mr. Pegram's discussions of the papers "What is the Life of an Iron Railroad Bridge?" and "American Railroad Bridges," as well as reviews in some of the text books, indicate that a uniform train load with a single concentration may produce strains generally within 10% of the engine specified. This would seem about as close as the equivalent typical engine, generally used, would come to the actual load, and, if so, much labor would be saved by the adoption of the single concentration method. The question, therefore, would appear to be, not whether the method of wheel concentration should be continued, but what equivalent load should be used? It should give results approximately equivalent to those obtained in practice; but at the same time the method should be so concise as to permit of its insertion in a specification.

It is, perhaps, unnecessary to discuss further the fatigue formula. The Launhardt formula, which is now used by a number of the larger trunk lines, is a precise interpretation of precise experiments which are never precisely repeated in practice. A study of these same experiments indicates that if we consider the destructive effect of live

Mr. Seaman. strain as about twice that of dead strain, we will obtain even better general results than by the Launhardt formula, and since this general ratio of 2 to 1 has long been adopted in the better bridge practice, there is no justification for the use of the Launhardt formula.

Mr. Snow. J. P. Snow, M. Am. Soc. C. E. (by letter).—The assumptions that it is necessary to make on the matters of typical loads, impact, proper unit strains, fatigue formulas, structural motion, personal predelictions as to column formulas, assessment of riveted members, proper differences for soft and medium steel and many other elements of bridge design which depend on the personal views of the designer, render any hair-splitting nicety absurd, either in computing the strain-sheet or in applying the various rules for deducing sections after the strain-sheet is determined.

The railroad official who buys bridges and who does not know how to design them must have a method which he can trust for specifying what he wants, both as to loading and the translation of that load into the final sections of the bridge members.

Without question, a definite engine loading is understood much more readily by such an official than a table or diagram of uniform loads; and Cooper's specification or some other generally accepted one is confidently referred to by him with the supposition that it gives a clear-cut and sufficient set of rules to guide all competing bridge builders in translating the prescribed load into the desired bridge.

The usefulness of the engine loading disappears if the bridge buyer designs his own bridges. The writer has used the uniform-load method exclusively for the last 15 years, both for designing new bridges and examining old ones, and finds it in every way satisfactory and ample, in all cases, to cover the whole ground.

The competing bridge builders who must prepare designs and tenders for the various loadings and specifications offered have the worst of it.

They must find the absolute maximum strains, or the consulting engineer to whom the tenders are referred will throw out their plans; and they must not make such liberal assumptions as will cause them to use more metal than is absolutely needed, else their keener competitors will underbid them. It is a case of full and exact figuring to the last possible permutation with them on every specification offered, and all this with the dreary knowledge that not more than one bid in a dozen will result in an actual contract.

Unless the bridge buyers can be brought to specify uniform loads for their bridges it is difficult to see how the overworked builders can be helped. One set of loads for moments and another for shears, as some have recommended, can hardly appeal to their sense of consistency. A uniform load plus a single rolling load, as many ad-

vocate, will not afford them much chance for comparison with familiar structures or train weights. Mr. Snow.

A single set of uniform loads giving the same center moment as the maximum wheel loads assumed serves as the basis of all the writer's computations and seems to him to be the simplest and most easily comprehended of any that he has seen advocated. The floor-beam load is, of course, the panel length multiplied by the load per foot corresponding to a span equal to twice the panel length. This load is used as a head load in getting the web shears, the following panel loads being those used for the chord moments. By neglecting the partial load ahead of the point under consideration an excessive shear is obtained, but many good designers advocate this practice, and, in the writer's opinion, it is justified, from the fact that under a fast moving train the web members appear to be much more disturbed than the chords.

Shear on longitudinal girders is obtained by using 57% of the total live load for the end reaction and 19% for the center shear. Intermediate shears are proportional to these. The actual maximum shear produced by a set of wheel loads expressed graphically is a flat curve; but the straight line given by the above percentages covers its maximum points and gives a small surplus at the quarter point of the span.

A system of loads for all spans cannot be properly made up from a single class of engines. Heavy passenger engines with two driving axles, or tank shifters on two axles close together, must be used for spans of less than 20 ft., while consolidation engines will govern on long spans. Car loads are so heavy at present that the customary legend of two engines followed by a certain train load may well be abandoned, and a continuous string of typical engines used in obtaining moments.

It appears to the writer that if prominent bridge engineers could bring themselves to re-write their specifications, giving simple tables or curves of the uniform loads per foot corresponding to various sets of typical wheel concentrations, and give rules for using these loads for all cases, that the bridge buyers would quickly fall into line and the situation would be relieved.

The speaker would vote unreservedly in favor of discontinuing the method of wheel-concentrations in determining the stresses in railroad bridges.

Edwin Thacher, M. Am. Soc. C. E. (by letter).—The first specification the writer ever saw giving wheel concentrations as a basis for the calculation of stresses due to live load was issued by the Cincinnati Southern Railroad about 25 years ago. Previous to that time, uniform loads had invariably been used in calculations; unit stresses being varied arbitrarily to suit length of span and location of member. Mr Thacher.

Mr. Thacher. This specification gave a live-load diagram of two 10-wheel engines, 36 tons in 12 ft. and 66 tons in 50 ft., followed by 1 820 lbs. per lineal foot. The writer was then employed as Assistant Engineer for the Louisville Bridge and Iron Company, of Louisville, Ky., of which the late Albert Fink, Past-President, Am. Soc. C. E., was President. The specification was received by the morning's mail, together with a letter inviting proposals for several bridges at an early date.

The combined talent of the office was assembled, and that specification was thoroughly discussed. It was voted unanimously that the specification was absurd; that an engine of such weight had never been heard of, and would never be used on the Cincinnati Southern Railroad or any other; that to design bridges for such a load was a reckless waste of money, and that to specify such a load was an outrage. After this feeling had had time to cool off, it was fully realized that this load must be used in calculations, and that the estimates and strain sheets must be prepared speedily, and the all-important question was how to do it. This was before the days of moment tables, or moment diagrams, and before any investigations had appeared giving the exact location of load for the maximum stress in any member.

Late in the afternoon of that day, R. Montfort, M. Am. Soc. C. E., now Chief Engineer of the Louisville and Nashville Railroad, suggested a method of calculation that was at once adopted, and which gave results near enough for all practical purposes. The first driver was placed over a panel point and the panel weights resulting from such location were used in the calculation, and, for double intersection bridges, in which the stresses are somewhat ambiguous at best, not much improvement can be made on that method to-day.

Specifications giving wheel concentrations, once introduced, came rapidly to the front, and in a short time all the railroads were in line, each with a different kind of load, and the bridge companies shouldered their burden, which they have continued to carry ever since.

The writer has been satisfied for many years past that the method of wheel concentrations in the calculation of bridge stresses was a waste of time, and has done what he could to correct the evil. In the specifications of the Keystone Bridge Company of 1887 and 1890 are given tables for equivalent uniform loads for engines of ten different weights, from 44-ton narrow gauge, to 112-ton decapod, and for spans from 10 ft. to 300 ft. in length. For spans of 70 ft. and less (70 ft. being about the maximum length of plate girders used at that time) the uniform loads are such as will give a maximum center moment; and for spans of 75 ft. and upward, such as will give a maximum end shear. These tables have been quite extensively used, and give results practically the same as the wheel loads, and in one-tenth the time. To prepare a moment table for a new loading is some labor, but when

once prepared, the equivalent uniform load for any span can be found, by aid of the slide rule, in a few minutes. Mr. Thacher.

It was the practice of the writer for some years to calculate bridges, using wheel concentrations, and check by the method of uniform loads, but gradually he came to use uniform loads altogether, except in special cases, generally confined to cases in litigation or reconstruction.

It appears to the writer that the railroad companies would, on proper representation, agree on four or five typical loads, ranging from light to the heaviest in use, some one of which they would be willing to adopt for their bridges. By an expert bridge engineer these could soon be tabulated for convenient use, giving the moments and shears required in girder calculations, and the equivalent loads to be used in the calculation of trusses, thus saving an immense amount of useless labor repeated over and over again. The writer is satisfied that this result will never be accomplished by the efforts of any one engineer, however meritorious his scheme, but believes it can be accomplished by a Special Committee of this Society.

Few engineers believe that Launhardt's formula should be used in designing bridges on account of any fatigue of the material due to repeated application of load, but the writer believes that a modification of this formula is the best thing yet devised for taking account of impact in the distribution of bridge material.

The writer prefers the form, $a\left(1+\frac{\text{Min.}}{\text{Max.}}\right)$. Cooper's specification reduces to $b\div\left(1+\frac{\text{L. L.}}{\text{Max.}}\right)$; and both give practically the same result. It is generally conceded that live load has a worse effect on a bridge than dead load, and the above formulas assume that it has double the effect, but they can be varied to suit any ratio desired, and the results are consistent with each other. The writer could never see any merit in the arbitrary rule-of-thumb methods that will add 25% for girders 25 ft. long, and 15% for girders 25 ft. 1 in. long; 50% to this and 10% to that; and he will continue to advocate the above modification of Launhardt's formula until something better is proposed.

J. A. L. Waddell, M. Am. Soc. C. E. (by letter).—In 1891 the writer presented to this Society a paper entitled "Some Disputed Points in Railway Bridge Designing,"* in which he advocated the adoption of a few standard train loads for railroad bridges, instead of the numerous ones then in use, offering instead a set of live loads for discussion, and urging that the "Equivalent Uniform-Load Method" be adopted instead of the tedious, burdensome, hair-splitting and heart-breaking method of "Wheel Concentrations." Mr. Waddell.

The thorough discussion which this paper received, and the favor-

* *Transactions*, Am. Soc. C. E., Vol. xxvi, p. 77.

TABLE No. 1—(*Continued*).

300.....		Batter brace.....	0.4 danger.	1.4 danger.	0.3 danger.	0.7 safety.	1.5 danger.	1.3 danger.
		1st main diag.....	0.6 "	0.9 "	0.2 safety.	0.1 "	0.8 "	0.8 "
		2d "	0.6 "	0.0 "	1.1 "	0.7 danger.	0.3 "	0.5 "
		3d "	0.9 "	0.8 safety.	1.8 "	1.5 "	0.6 safety.	0.1 safety.
		4th "	1.0 "	2.0 "	2.9 "	2.3 "	1.9 "	1.0 "
		5th "	0.7 "	4.1 "	4.7 "	2.8 "	4.1 "	2.5 "
		1st counter.....	0.4 safety.	7.5 "	7.5 "	3.0 "	7.4 "	4.7 "
		2d "	1.9 "	12.5 "	11.2 "	2.5 "	12.9 "	8.0 "
	1st top chord.....	.....	0.1 danger.	1.1 danger.	0.1 danger.	0.7 safety.	1.4 danger.	1.4 danger.
	2d "	.....	0.5 safety.	0.5 "	0.4 safety.	1.0 "	1.2 "	1.4 "
	3d "	.....	0.7 "	0.3 "	0.4 "	1.6 "	0.6 "	0.9 "
	4th "	.....	0.7 "	0.3 "	0.3 "	1.8 "	0.4 "	1.0 "
	5th "	.....	0.6 "	0.4 "	0.1 danger.	1.8 "	0.3 "	1.3 "
400.....		Batter brace.....	0.1 danger.	1.3 "	0.3 "	0.7 "	1.5 "	1.1 "
		1st main diag.....	0.2 "	1.1 "	0.0 safety.	0.4 "	1.2 "	0.8 "
		2d "	0.4 "	0.7 "	0.4 "	0.2 danger.	1.0 "	0.7 "
		3d "	0.5 "	0.3 "	0.8 "	0.8 "	0.7 "	0.5 "
		4th "	0.8 "	0.1 safety.	1.2 "	1.4 "	0.3 "	0.2 "
		5th "	0.9 "	0.6 "	1.8 "	2.0 "	0.3 safety.	0.3 safety.
		6th "	1.1 "	1.4 "	2.5 "	2.6 "	1.1 "	0.9 "
		7th "	1.0 "	2.5 "	3.5 "	3.2 "	2.3 "	1.8 "
		1st counter.....	0.8 "	4.1 "	5.0 "	3.9 "	3.6 "	2.7 "
		2d "	0.7 "	6.0 "	6.6 "	4.7 "	5.6 "	4.0 "
		3d "	1.3 safety.	10.5 "	10.5 "	4.9 danger.	9.0 "	6.1 "
	1st top chord.....	.....	0.1 "	1.1 danger.	0.1 danger.	0.7 safety.	1.5 danger.	1.2 danger.
	2d "	.....	0.3 "	0.9 "	0.1 safety.	0.8 "	1.4 "	1.1 "
	3d "	.....	0.6 "	0.7 "	0.2 "	0.9 "	1.3 "	1.1 "
	4th "	.....	0.6 "	0.6 "	0.2 "	1.3 "	1.2 "	1.1 "
	5th "	.....	0.6 "	0.6 "	0.2 "	1.4 "	0.9 "	0.9
	6th "	.....	0.6 "	0.6 "	0.1 "	1.4 "	0.8 "	0.9 "
	7th "	.....	0.6 "	0.6 "	0.1 danger.	1.4 "	0.8 "	1.1 "
	Bottom chord.....	.....	0.1 danger.	1.3 "	0.3 "	0.7 "	1.5 "	1.1 "

TABLE No. 1.—Showing Percentages of Error in Stresses, found by Equivalent-Uniform, Single-Excess and Double-Excess Methods of Loading.

Span length.	Chord members.	Web members.	Class Z. Equivalent-uniform system.	Class Z. Single-excess system.	Class Z. Double-excess system.	Class U. Equivalent-uniform system.	Class U. Single-excess system.	Class U. Double-excess system.
100 ft.		Batter brace	0.9 danger.	4.6 safety.	0.9 safety.	1.8 danger.	7.5 safety.	1.2 danger.
		1st main diag.	6.1 safety.	20.2 "	8.5 "	4.5 safety.	26.0 "	5.3 safety.
		1st counter.	19.9 "	60.0 "	39.0 "	13.5 "	66.0 "	39.0 "
	1st top chord.		3.7 "	9.6 "	0.9 "	4.9 "	15.0 "	0.8 danger.
	Bottom chord.		0.9 danger.	4.6 "	0.9 "	1.8 danger.	7.5 "	1.2 "
150 ft.		Batter brace	1.6 "	0.1 danger.	0.1 danger.	1.1 "	0.9 "	1.7 "
		1st main diag	0.3 "	4.3 safety.	3.1 safety.	0.6 "	6.0 "	1.3 safety.
		2d "	3.2 safety.	12.6 "	9.0 "	1.4 safety.	15.3 "	6.0 "
		1st counter.	10.2 "	30.1 "	17.0 "	7.2 "	35.0 "	12.8 "
	1st top chord.		1.1 "	2.8 "	1.7 "	0.6 "	2.5 "	1.4 danger.
	2d "		1.3 "	2.6 "	0.3 "	2.6 "	4.7 "	1.6 "
	Bottom chord.		1.6 danger.	0.1 danger.	0.1 danger.	1.1 danger.	0.9 "	1.7 "
200 ft.		Batter brace	1.1 "	1.1 "	0.3 "	0.2 "	0.8 danger.	1.7 "
		1st main diag.	1.0 "	0.7 safety.	1.2 safety.	0.7 "	1.3 safety.	0.3 "
		2d "	0.5 "	3.4 "	3.4 "	2.8 "	2.6 "	0.1 "
		3d "	0.8 safety.	7.8 "	6.7 "	1.3 "	8.9 "	4.1 safety.
		1st counter.	5.9 "	18.7 "	14.6 "	2.6 safety.	20.7 "	11.0 "
		2d "	11.8 "	35.5 "	22.2 "	7.5 "	40.3 "	17.0 "
	1st top chord.		0.4 "	0.4 "	0.9 "	0.7 "	0.1 "	1.3 danger.
	2d "		0.7 "	0.7 "	0.7 "	1.6 "	1.1 "	1.0 "
	3d "		0.6 "	0.6 "	0.2 danger.	2.2 "	1.7 "	1.4 "
250 ft.		Batter brace	0.8 danger.	1.4 danger.	0.3 "	0.4 "	1.3 danger.	1.5 "
		1st main diag.	0.9 "	0.4 "	0.5 safety.	0.3 danger.	0.3 "	0.7 "
		2d "	0.9 "	0.9 safety.	1.7 "	1.3 "	0.8 safety.	0.1 "
		3d "	0.9 "	2.5 "	3.1 "	2.1 "	2.7 "	1.1 safety.
		4th "	0.3 "	5.5 "	5.5 "	2.4 "	5.9 "	3.1 "
		1st counter.	1.9 safety.	11.1 "	9.9 "	1.4 "	11.8 "	6.8 "
		2d "	5.9 "	20.9 "	17.0 "	1.7 safety.	22.9 "	13.0 "
	1st top chord.		0.4 "	0.2 danger.	0.8 "	0.6 "	1.1 danger.	1.5 danger.
	2d "		0.5 "	0.1 "	0.6 "	1.1 "	0.6 "	1.3 "
	3d "		0.7 "	0.1 safety.	0.5 "	1.6 "	0.0 "	1.2 "
	4th "		0.5 "	0.1 danger.	0.1 danger.	1.9 "	0.3 safety.	1.5 "

Mr. Waddell.

able opinions of most of those who discussed the questions, and of the technical press, induced the writer to carry farther his investigations and to make an earnest endeavor to establish a few standard live loads and the simpler method of computing stresses.

The reviews in the technical press started a series of papers by the writer and others, which were printed in the *Railroad Gazette* and *The Engineering Record*.

These showed conclusively that the "Equivalent Uniform-Load Method" gives results which are accurate enough for all practical purposes, and which are more accurate than those which are obtained by either the "Single-Concentration Method" or the "Double-Concentration Method." Concerning the accuracy of this, there appears to be still some doubt in the minds of a few members of the Society, as shown by opinions expressed in this discussion, so the writer gives herewith certain tabulated results of computations made some years ago for the special purpose of comparing the three alternative methods with the so-called exact method, all tabulated figures having been checked by two computers working entirely independently of each other. The span assumed for comparison was one of 200 ft., which is not far from the average span length for truss bridges in America; and the live-loads adopted are those of Classes *Z* and *U* of the writer's "Compromise Standard System of Live Loads for Railroad Bridges."

For trusses, the equivalent uniform load used in each case is an average of those figured for each panel point. This, for nearly all spans, is almost exactly the same as the equivalent for the quarter points.

TABLE No. 2.—Showing Percentages of Error in Moments at Centers of Plate-Girder Spans, Found by Equivalent-Uniform, Single-Excess and Double-Excess Systems.

Span length. Feet.	Class *Z*.			Class *U*.		
	Equivalent uniform system.	Single-excess system.	Double-excess system.	Equivalent uniform system.	Single-excess system.	Double-excess system.
15....	none.	14.0 safety.	14.0 safety.	none.	19.9 safety.	19.9 safety.
20....	"	7.8 "	7.8 "	"	11.8 "	11.8 "
25....	"	4.2 "	4.2 "	"	5.3 "	5.3 "
30....	"	2.1 "	72.1 "	"	1.2 "	1.2 "
35....	"	0.7 "	0.7 "	"	1.3 danger.	1.3 danger.
40....	"	0.5 "	0.5 "	"	2.3 "	2.3 "
45....	"	0.3 "	0.3 "	"	2.8 "	2.8 "
50....	"	0.0 "	0.0 "	"	3.2 "	3.2 "
60....	"	0.4 danger.	0.4 danger.	"	2.7 "	2.7 "
70....	"	0.1 safety.	0.1 safety.	"	2.2 "	2.2 "
80....	"	0.0 "	0.0 "	"	1.9 "	1.9 "
90....	"	0.1 danger.	0.1 danger.	"	1.7 "	1.7 "
100...	"	0.9 "	0.9 "	"	1.2 "	1.2 "

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TABLE No. 3.—Showing Percentages of Error in End Shears, Found by Equivalent-Uniform, Single-Excess, and Double-Excess Systems.

Span length. Feet.	Class *Z*.			Class *U*.		
	Equivalent uniform system.	Single-excess system.	Double-excess system.	Equivalent uniform system.	Single-excess system.	Double-excess system.
15....	none.	1.9 safety.	22.5 danger.	none.	45.4 safety	3.0 danger.
20....	"	6.0 "	16.7 "	"	33.3 "	10.0 "
25....	"	7.6 "	13.2 "	"	22.8 "	12.3 "
30....	"	5.8 "	12.8 "	"	18.1 "	13.3 "
35....	"	2.8 "	13.9 "	"	14.3 "	14.3 "
40....	"	1.7 "	13.8 "	"	12.0 "	14.4 "
45....	"	0.7 "	13.5 "	"	10.5 "	14.1 "
50....	"	0.6 "	12.5 "	"	9.45 "	13.5 "
60....	"	0.5 "	7·8 "	"	7.9 "	8.4 "
70....	"	1.5 danger.	6.8 "	"	4.0 "	7.6 "
80....	"	3.2 "	6.6 "	"	0.7 "	7.9 "
90....	"	4.3 "	6.4 "	"	1.5 danger.	8.0 "
100...	"	5.1 "	6.4 "	"	2.7 "	7.9 "

In these tables the best possible showing was made for both the "Single-Concentration Method," and the "Double-Concentration Method," as an inspection will show.

The single concentrated load for each class was found by ascertaining, for each main diagonal and each chord section of a 150-ft., a 300-ft., and a 500-ft. span, the single concentrated excess load, which, in addition to the uniform car-load, would produce the same shear or moment as that found by the wheel-concentration method. These excesses for each span were then averaged, and, finally, the average of these averages was taken as the most nearly correct excess load for main-truss members. The results were, respectively, for Classes *Z* and *U*, 39 372 lbs. and 88 368 lbs.

Next, the average single concentrations for all plate-girder spans between 20 ft. and 100 ft. for both classes were determined, and found to be, respectively, 24 577 lbs. and 51 750 lbs. Now, as the floor system is quite as important as the trusses, it was deemed proper to average these concentrations with those found for main-truss members, in order to determine the final single concentration for each class, thus making the said final concentrations, in round numbers, 32 000 lbs. for Class *Z* and 70 000 lbs. for Class *U*.

On applying these to both floor systems and trusses, the errors found were so great as to show conclusively that no single concentration can be assumed which, combined with the constant car load, will give results that even approximate to those found by the concentrated wheel-load method.

It became necessary, therefore, to adopt one concentration for the floor system and another for the trusses, in order to make any show-

Mr. Waddell. ing at all for the system. For Class *Z* there were used 25 000 lbs. for the floor system and 40 000 lbs. for the trusses, and for Class *U* 50 000 lbs. and 90 000 lbs., respectively. On applying these, it was found necessary to make still another compromise, viz., to use the truss concentration in computing end shears in plate-girder spans and in floor systems.

Adopting these compromises, and making calculations thereby, gave the percentages of error shown in Tables Nos. 1, 2 and 3, in the columns headed "Single-Excess system."

The double concentrated loads for the two classes were found by making some extensive calculations. These loads varied from 25 000 lbs. for Class *Z* to 50 000 lbs. for Class *U*. By using these, the percentages of error in Tables Nos. 1, 2 and 3, under the headings "Double-Excess System," were ascertained.

A study of Table No. 1 shows that the equivalent uniform-load method is much more accurate for main-truss members than either the single or the double-concentration method, and, referring to Table No. 2, it is seen that, while the equivalent-uniform load gives exact results (or as nearly so as a due consideration for regularity in the curves will permit), both of the other methods, especially for short spans and stringers of ordinary length, give large errors, notwithstanding the fact that there were adopted different concentrations for floor systems and trusses.

In order to eliminate entirely the personal equation from these tables, Table No. 4 was prepared by collecting the greatest summations of percentages of error for both chords and webs, counters being ignored:

TABLE No. 4.—Showing Variations in Percentages of Error, as given by Equivalent-Uniform, Single-Excess and Double-Excess Systems.

Span Length. Feet.	Members.	Class *Z*.			Class *U*.		
		Equivalent uniform system.	Single-excess system.	Double-excess system.	Equivalent uniform system.	Single-excess system.	Double-excess system.
100..	Chord members.	4.6	5.0	.....	6.7	7.5	0.4
	Web "	7.0	15.6	7.6	6.3	18.5	6.5
150..	Chord "	0.2	0.2	1.4	2.0	2.2	0.2
	Web "	4.8	12.7	9.1	2.5	14.4	7.7
200..	Chord "	0.3	0.3	1.1	1.5	1.6	0.4
	Web "	1.9	8.9	7.0	2.6	9.7	5.8
250..	Chord "	0.3	0.3	0.9	1.3	1.4	0.3
	Web "	0.6	6.9	5.8	2.8	7.2	4.6
300..	Chord "	0.8	0.8	0.5	1.1	1.1	0.5
	Web "	0.6	5.5	5.0	3.5	5.6	3.8
400..	Chord "	0.5	0.5	0.3	0.7	0.7	0.3
	Web "	1.0	3.8	3.8	3.9	3.9	2.9

When the extreme errors were of opposite kinds, their arithmetical sum was taken, but when they were of the same kind their numerical difference was taken. Mr. Waddell.

A study of Table No. 4 shows that, as far as chord stresses are concerned, there is but little to choose from between the three methods, but that, in respect to web members, the equivalent uniform-load method is very much more accurate than either the single-excess or the double-excess method.

Summarizing, it is evident that the following have been proved:

1st. The equivalent uniform-load method, for all bridges of types sanctioned by the best modern practice, gives stresses agreeing sufficiently well for all practical purposes with those found by the concentrated wheel-load method.

2d. That there is no combination of a constant car load per lineal foot and a single concentrated load which will give, for all members of trusses and floor systems, stresses which will agree even approximately with those found by the concentrated wheel-load method.

3d. That even if for any system of loading two separate single concentrations be adopted, one for floor systems and the other for main-truss members, the said single concentrations cannot be adjusted so as to give results agreeing sufficiently well for practical purposes with those found by the concentrated wheel-load method.

4th. That if two engine excesses placed about 50 ft. apart at panel points be adopted in connection with a constant car load per lineal foot, the stresses obtained thereby in members of floor systems and trusses do not agree sufficiently well, for practical purposes, with those obtained by the concentrated wheel-load method.

Now, on the strength of this showing, what is the use of talking any more about using a constant car load per lineal foot headed by either one or two concentrations? Either of the last two methods requires, in making calculations of stresses, fully double the time and labor required by the equivalent uniform-load method, and is by no means as accurate.

There is no method of computing live-load stresses so simple and easy as that which involves the use of properly figured uniform loads, so, why not adopt it for good and all, if we are to abandon in future the use of the "Wheel-Concentration Method"?

And surely the time has come to abandon the latter method forever, if one may judge by the unanimous opinion of those who took part in the oral discussion of the question.

In respect to Mr. Morison's proposed method, it must be evident to all that instead of simplifying matters it would complicate them worse than ever.

The objections which railroad engineers offer to using live loads which they cannot understand readily, and which, at least in their minds, do not approximate actual train loads, is well founded.

Mr. Waddell. Let each railroad engineer who desires to have a bridge designed choose a combination of two locomotives and a train of cars of indefinite length weighing a certain number of pounds per lineal foot, and, on the same diagram upon which this loading is drawn to scale, let there be drawn also the curves for equivalent loads and for end shears in plate-girder spans; and let there be written thereon a note to the effect that the curves are to be used instead of the actual wheel concentrations.

But, instead of each engineer having a live load of his own, why not provide a variety of live loads differing from each other by uniform gradations from which anyone can choose a load to suit his requirements?

"In November, 1892, the author sent a circular-letter to all the chief engineers of railroads in the United States and Canada who were members (in any grade) of the American Society of Civil Engineers, and to every other member of that society connected with or specially interested in the designing, building or operating of railroad bridges. This letter solicited a ballot on certain 'Disputed Points in Railway Bridge Designing,' foremost among which were those of standard live loads and a simple equivalent method for computation. The number of responses received was as great as could have been expected; and the result was that about 82% of those who voted favored, and 18% opposed, the adoption of 'A Standard System of Live Loads for Railway Bridges' similar to that proposed by the author. Eighty-two per cent. also of those who voted were in favor of abandoning the 'Concentrated Wheel-Load Method,' and 18% were in favor of retaining it. Of the former, 78% favored the 'Equivalent-Uniform Load Method,' and 22% were in favor of either the 'Single' or the 'Double Concentration Method.' A number of gentlemen who responded made valuable suggestions in respect to the standard system of live loads propounded, and by the aid of these the author prepared a proposed 'Compromise Standard System of Live Loads for Railway Bridges,' and submitted the same, as before, for final ballot in May, 1893.

"The number of replies received showed that great interest was taken in the question; and the result of the ballot was 90% in favor and 10% opposed to the proposed standard.

"Next the pamphlet was published and distributed quite generally among those engineers interested in the subject of bridges, a copy being sent not only to every one who had replied to the ballots, but also to every railroad chief engineer in the United States, Canada and Mexico whose address was given in *Poor's Manual*. To these chief engineers there was also sent another circular-letter with a ballot that read as follows:

"'I $\left\{\begin{matrix}\text{agree}\\\text{do not agree}\end{matrix}\right\}$ to use the "Compromise Standard System of Live Loads for Railway Bridges" when calling for bids on railroad-bridge work, or when having plans prepared for railroad bridges.'

"'I $\left\{\begin{matrix}\text{agree}\\\text{do not agree}\end{matrix}\right\}$ to specify that the "Equivalent Uniform-Load Method" is to be used in computing stresses in the bridges that are to be designed for my road.'

"Over one hundred chief engineers thus addressed voted in favor of both propositions, and very few were opposed."

Mr. Waddell.

The Compromise Standard System of Live Loads for Railroad Bridges has now been in use for six years, and all who have used it and its equivalents endorse heartily both the loads themselves and the equivalent method of calculating stresses.

Unfortunately, locomotives and cars have both grown somewhat heavier since the writer's pamphlet was issued, so that it is now necessary to add to the live loads "Class *S*," for which the axle loads would be as follows:

Pilot axle	22 000 lbs.
Driving axles	43 000 lbs. each.
Tender axles	25 000 lbs. each.
Cars	4 400 lbs. per lineal foot.

Alternative axle load, 54 000 lbs. each, on two axles only.

The equivalent curves can be drawn on the diagrams by eye, correctly enough for all practical purposes, by simply extending upward the spacing of the curves.

There is one point in connection with all these equivalent curves to which the writer desires to call attention, viz., the equivalent loads which they represent were figured for cars preceding as well as following the locomotives, in the case of chord stresses.

The writer desires also to call attention to the short method, evolved by him some eight years ago, of computing for any truss, by one simple graphic diagram and by the use of a slide rule, all the main-member stresses, both direct and reverse.

The method is as follows:

Assume that there is an upward reaction at one end of the span equal to 100 000 lbs., caused by an unknown load placed at the first panel point from the other end of the span, then find graphically the stress in each web-member from end to end of span, caused by this assumed upward reaction. Then calculate the value of the live-load reaction for the maximum stress in each web-member by means of the slide rule and the following formula and table in which n is the number of panels in the span, n' is the number of the panel point at the head of the train, counting from the loaded end of the span, and C is the coefficient of $\frac{L}{n}$.

Live-load reaction for the head of train at $n' = C \times \frac{L}{n}$.

n'	C	n'	C	n'	C	n'	C
1	1	7	28	13	91	19	190
2	3	8	36	14	105	20	210
3	6	9	45	15	120	21	231
4	10	10	55	16	136	22	253
5	15	11	66	17	153	23	276
6	21	12	78	18	171	24	300

Mr. Waddell. Then, still using the slide rule, find the greatest live-load stress in each web-member by the following equation:

$$\text{Stress required} = \text{Stress from Assumed Reaction} \times \frac{\text{Actual Reaction}}{\text{Assumed Reaction}}$$

Where the panels are divided, as in the Petit truss, and where inclined sub-posts are used, the tensile stress in the upper half of each main diagonal thus found will have to be corrected by subtracting therefrom a stress equal to $\frac{L}{2}$ Sec. A, where A is the inclination of the diagonal to the vertical. But when inclined sub-ties are used instead of inclined sub-posts, the correction just referred to will apply only to the compressive stresses in the lower halves of the main diagonals. The reason for making this correction, as will be at once evident to any one who is accustomed to finding stresses in Petit trusses, is that the method above outlined ignores the subdivision of the panels when ascertaining by graphics the stresses caused by the assumed upward reaction.

The writer's sole reason for again bringing forward this quick method is that he would like bridge engineers to learn to use it and thus save themselves from wasting much valuable time and energy, as it cuts down the work of computing web stresses to a very small percentage of the amount usually expended thereon.

In conclusion, it appears from the opinions advanced in the oral discussion, that the time has come to drop from engineering practice and to relegate to text books, where it can still serve as an excellent example of mental gymnastics, the pseudo-exact, laborious and tedious method of Wheel Concentrations, and to adopt instead some simpler methods; consequently, the writer herewith proposes that the "Compromise Standard System of Live Loads for Railroad Bridges" and the "Equivalent Uniform Load Method" of computing stresses be adopted by the profession for general practice.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
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PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications

STREAM CONTAMINATION AND SEWAGE PURIFICATION.

An Informal Discussion at the Annual Convention, June 28th, 1899.*

SUBJECTS FOR DISCUSSION:

(*a*) "Should Stream Contamination by the Sewage of Cities be absolutely prohibited by law?"

(*b*) "Should the Purification of the Sewage of Cities be compulsory; and is this feasible for large cities?"

(*c*) "Is Filtration the coming solution of the Pure-Water Question for Cities?"

By Messrs. R. E. McMATH, W. C. PARMLEY, GARDNER S. WILLIAMS, PETER MILNE, KENNETH ALLEN, JAMES OWEN, CHARLES G. DARRACH, E. W. HARRISON, P. A. MAIGNEN, L. L. TRIBUS, PALMER C. RICKETTS and JAMES S. HARING.

R. E. McMATH, M. Am. Soc. C. E.—This question has been sub- Mr. McMath.
divided into three topics, the first being: "Should Stream Contamination by the Sewage of Cities be absolutely prohibited by law?"

This question may be answered in the affirmative on the ground of the common-law principle: "Each must so use his own as not to injure any other." No individual can discharge filth or other objectionable matter into a ditch or drain which traverses the field of his neighbor. If he attempts to do so the court will afford the injured party a remedy. If a city, town or village defiles a stream with sewage, the

*The discussion of this subject, for which no formal paper was presented, is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication. *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to September 23d, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

Mr. McMath. law is an already existing remedy, and has been applied in many cases which could be cited.

The presentation of this topic for discussion by this Society does not imply any doubt as to this recognized law, but, perhaps, is intended to call the attention of engineers to its existence and to the necessity for its extension to questions which, in this country, may arise between the people of adjacent States, if the defiled stream is a boundary, or passes from the territory of one State into that of another. That is to say, is there not a necessity for a general law which will embrace all States, and will settle all questions which arise between States?

Several of the States do exert their powers to prevent contamination of streams by the cities under their jurisdiction, but there is, as yet, no statutory provision to protect against contamination by the sewage of cities in other States. If, by the indifferent or mistaken policy of one State, its cities, great or small, are allowed to defile the streams, must the people of other States suffer for lack of adequate remedy?

The established law referred to has not been enforced in many cases where the transgressor is a city, but the transgression being for a time condoned does not establish a right to transgress for all time. As the country increases in population, the pollution of streams will increase and will quickly pass beyond possible toleration.

As the speaker views the matter, a law more imperative than any statute should control every engineer who has to do with the sewerage plans of cities; and he should foresee and provide for the disposal of sewage eventually, if not immediately, in another manner than by discharge in crude form into streams. Anything short of this is not worthy to be called a plan for a sewerage system.

The City of Chicago has for years been preparing a channel for the discharge of its sewage through its back door, so that this sewage will pass the front doors of other cities, and some of these other cities are not well pleased by the prospect, especially since the demonstration that certain diseases are water working and that the pathogenic bacteria are tenacious of life. The topic under discussion would be an important one if Chicago did not exist, but the Chicago case gives much emphasis to the topic at the present time, for it is a scheme to do permanently, and on a large scale, what everyone must admit should not be done at all—a scheme which is to-day without an avowed designer and almost without a defender.

There was a time when many believed that the water of a river purified itself. Others firmly believed that dilution to a point beyond the reach of refined chemistry must render such water innocuous. The Chicago scheme had its origin when these beliefs were held. This scheme, involving the expenditure of more than $30 000 000, has progressed, notwithstanding the passing of the faith upon which it was founded, and now the only plea that is advanced for its consummation

is the fact that $28 000 000 and more have been expended on it and it is now too late for objectors to state their objections. Mr. McMath.

This brings up the consideration of the second topic: "Should the Purification of the Sewage of Cities be Compulsory; and is this feasible for Large Cities?"

The answer to the first part of this proposition follows that given for the preceding topic. If streams are not to be contaminated, their sewage must be purified; for water is the only known vehicle that can carry the wastes of human life and industry away from our homes and shops. That such can be purified so that the effluent may be entirely unobjectionable has been demonstrated beyond occasion for further discussion. How to do it, in each case that may arise, is the work of engineers, to plan, execute and operate.

There is no casuistry in the assertion that in this matter the end justifies the means, even when the difficulties surrounding the end require large and many means.

As to its feasibility for large cities there are too many old-world examples of successful solution for American engineers to hesitate as to the answer. It can be done, and we are the men who can do it.

The third topic comes in to round out the discussion: "Is Filtration the coming solution of the Pure-Water Question for Cities?"

Asked in connection with the preceding topic, the question may be taken as a suggestion that the sufferer from sewage contamination of water supply must protect himself, and thus relieve his up-stream neighbor of his obligation.

To meet this suggestion it is sufficient to mention the fact that the sufferers do not all live in cities, and are not able to protect themselves. There are evils from such contamination not measurable by the results of a bacteriological examination for specific microbes. There is also a contamination of water by the drainage of lands outside of cities that sooner or later will justify and even compel resort to filtration of the water supply of cities.

The doctors may say that they can cure a large percentage of cases of diphtheria, for instance, but the wise mother does not expose her children to infection on the chance that all cases in her family will be cured.

Therefore, filtration may promise a remedy for defiled waters, but that does not imply that the filtered water would not be better and safer had it not been contaminated.

W. C. Parmley, M. Am. Soc. C. E.—The question of stream pollution is one that concerns almost every city. If no pollution of the water courses is to be permitted, then all the sewage must be purified, and the question resolves itself into one of expense. Small towns, or those of larger size, which are favorably situated, may adopt some method of ultimate treatment at comparatively small cost, but in the Mr. Parmley.

Mr. Parmley. case of many, if not indeed most, of the larger cities, the expense involved by such treatment would be a very great burden, if not altogether out of the question. Insistence upon absolute freedom from pollution would make the interception of sewage and the overflow of storm water an impossibility.

The whole matter is one in which the ideal is set over and against the practicable—the desirable but extremely difficult to attain, against the less desirable but practically possible—but it is no more important that all the water we drink should be free from impurities than that the air we breathe and the food we eat should be pure and wholesome. To insist, however, upon a high degree of purity in a stream from which no drinking water is drawn, or at most only stock water, is to require heavy payment for very small benefits, and many streams have long since ceased to furnish even the latter supply.

The only other cases necessary to consider are those in which the odors arising from a polluted stream become unpleasant or harmful, or where the sight of turbid or colored waters or the refuse left by them along their banks is objectionable for æsthetic or sentimental reasons. The sanitary welfare of the people within the area affected is the ultimate and only condition to be insisted upon.

Even in the earlier development of our cities the natural drainage courses soon become so polluted by local discharge that no drinking supplies are likely to be taken from them, and it remains to be determined whether the pollution that has already occurred shall be removed or not. As most of this pollution is from factory wastes or from the washings of stables and privy vaults, it is a source of impurities nearly impossible to avoid. While the drainage area remains suburban, the cost of adequate sewerage would be too expensive to be borne by a sparse population or an isolated manufacturing company, and when the density of population becomes sufficient to make thorough sewerage possible, the soil and water-courses have already become so polluted that many years would be required to thoroughly drain them. Even then, as those who have had experience will testify, it is impossible to prevent entirely the clandestine discharge of some sewage into the streams. It follows, then, that the smaller courses which are not entirely taken into the sewerage system will have a considerable degree of pollution. Just the percentage that may be allowed is impossible to state, but it is dependent on the character of the polluting matter, the climatic and many other conditions. It is safe to say, however, that no stream should be permitted to become so befouled as to give off offensive effluvia and gases, thus showing that it contains a large amount of putrefying matter. Either, these streams should be diverted into the sewers, or the whole stream taken up in a large trunk sewer.

The question of permitting storm water from sewers to overflow into

water-courses is important in many cities. If such overflows are not allowed, the cost of the sewers thus required, over that where the overflow system is permitted, becomes an item of no mean proportions. The engineering department of the City of Cleveland has at the present time a contention with the Board of Park Commissioners regarding the overflow of storm water from a sewer into a stream; and if it is finally settled that the sewer cannot be permitted to overflow, the cost of the sewer will be about $250 000 more than if it were constructed with storm-water overflows. In another instance the local court has issued a restraining order to prevent the discharge of excess storm-water into a stream which is already highly polluted. The final settlement of the case is of great importance, as upon this decision will depend other important projects in the sewerage of that city. The court apparently ignored the important facts brought out in the trial, viz., that this overflow would not operate except in case of excessively heavy storms when the water would be discharged for perhaps 15 or 20 minutes, that these overflows would probably occur only a few times during the season, that the actual stream in time of flood would be nearly as impure as the water overflowed from the sewer, and that even if it were not, the time of maximum flood in the stream would be after the sewer had ceased to overflow and the stream would continue to flow with a large volume for many hours afterward, thus washing away the last traces of any fancied pollution. Mr Parmley.

If sewers are not allowed to discharge or overflow into natural water-courses, in many instances the double system of sewerage will become an absolute necessity. If the sanitary sewage be conveyed to a proper outlet through separate sewers and all the storm-water is to flow to the natural drainage channels, either through storm sewers or directly over the ground, the water discharged therefrom will usually be more foul than that which reaches them from the storm overflow structures of the combined sewers. As all engineers have probably noticed, the first washings of the street surfaces are extremely foul, and by the system of overflows this foul water is carried to the sewer outlet while a portion of the cleaner washings that occur later in the storm mixed with the normal volume of sewage will escape from the overflows into the stream.

If, however, the contaminating effect of street water be neglected and only the effect of sewage from the sanitary flow be considered, and if the entire volume of dry weather flow in the sewer be defined as sewage, in the first case above cited it was shown that the maximum degree of pollution, that is, when the sewer is just at the point of overflowing, is 1 of sewage to about 7 of street water, and when the structure is discharging its maximum volume of overflow the proportion is 1 of sewage to 30 of street water. Now, consider the degree of pollution of the stream into which this discharge is made and the

Mr. Parmley. proportion of pollution is much less in proportion to the volume of water already flowing therein.

It is a matter of the simplest observation to show that in many cases the water escaping from overflows in time of storm is not more foul than that in the swollen waters of the flowing stream. It being thus impossible to prevent the pollution of small courses flowing through thickly populated areas, it seems unwise to attempt to preserve in them a greater degree of purity than that required to prevent their contaminating the surrounding air, and when they are so short, or their velocity of flow is so rapid, that an outlet is reached at a point beyond which it is unnecessary to consider their impurities, and before those impurities have had an opportunity to decompose, the further expenditure of money, in order to preserve a high degree of purity, would be a needless waste.

The matter then suggests the following conditions with their appropriate treatment:

1st.—Streams or bodies of water from which potable supplies are drawn should be preserved in the purity required by the most advanced sanitary science, and if such purity can be obtained in no other way, the supply should be properly purified before being used.

2d.—Streams or ponds which have sluggish motion with consequent tendency to stagnation should be kept as free as possible from sewage pollution in order that their emanations may be less dangerous.

3d.—Streams in which the flow is so rapid or in which the courses are so short that stagnation and decomposition are impossible within the limits of time necessary to consider, may be used advantageously for the overflow of excess storm water, or as the direct outlet for a limited number of sewers.

Mr. Williams. GARDNER S. WILLIAMS, Assoc. M. Am. Soc. C. E.—It would be interesting to determine, if possible, where to draw the line as to the limit of the stream which is not used as a source of potable water. Nearly all streams flow into others or into lakes, and at some point drinking water is being taken from them.

An interesting case came under the speaker's observation, while he was connected with the water supply of Detroit, Mich., which involved the investigation of a serious typhoid epidemic in that city in 1892.

Port Huron is situated at the head of the St. Clair River and at the foot of Lake Huron. The distance by water from Port Huron to Detroit is about 60 miles, about 18 miles of which is through Lake St. Clair, where the current is very slow, and which acts as a monstrous settling basin, giving ideal conditions for sedimentation and the consequent purification of the water. The sewage of Port Huron is discharged into Black River, a very sluggish stream, which flows through the center of the city and receives sewage from both sides. The current in Black River is practically nothing, except in time of

freshet. No water for potable purposes is taken from this stream, but the Detroit and St. Clair Rivers and Lake St. Clair are sources of supply. Mr. Williams.

In 1890 Congress decided to improve the navigation of Black River, and work thereon began in 1891. Dredging was commenced at the bar at the mouth of the river, and was approaching that part of the river where the principal sewers discharge, when cold weather stopped the work. On April 16th, 1892, work was recommenced at a point about 300 ft. below the first bridge, and at that bridge two large sewers discharge into the river. Below that bridge it is reported that a depth of 12 ft. was dredged through an accumulation of sewage deposits which had been gathering there for years. The inspector's records of the dredging show that there were excavations in such material 8 ft. in depth. This material was put in dump scows, taken out into the St. Clair River and dumped at a point over the Grand Trunk Railroad tunnel, or approximately 60 miles above the intakes of the Detroit Water-Works, which are just below Lake St. Clair, and virtually at the head of the Detroit River.

The record of typhoid fever in Detroit at that time is interesting. There had been some deaths from typhoid early in the year, but between May 11th and May 28th, 1892, no deaths were reported. On May 28th there was one death. On June 5th there were four, and during the ensuing 25 days there were thirty-seven deaths from typhoid fever, the population of the city being then about 230 000. The death rate from typhoid in that year in Detroit rose as high as the typhoid death rate of any large American city, except Chicago, Pittsburg, Jersey City and St. Louis, in the past ten years. There were three years in which the death rate in Pittsburg and two in which that of Chicago had been higher.

It was somewhat difficult to trace the cause of this epidemic, and when the speaker began his investigations he was not aware of the operations of the Government in Black River in 1891 and 1892. He therefore did not look there for the pollution, but began examining the river and studying rainfall, the effect of thaws, the effect of ice in the river and the character of the water. None of the ordinary methods of accounting for the presence of typhoid was applicable in this case.

The precipitation during April had been heavy, but it had also been heavy at many other times when there was no typhoid. During that summer all the river cities had typhoid, and in all cases the disease appeared at about the same time. At Port Huron, while there had been a tremendous excess of typhoid late in the fall of 1892, there were only one or two cases in the spring of that year; this pointed to a cause at or below Port Huron.

Finally, the speaker's attention was called to the Government

Mr. Williams. dredging operations and to the place where the material was dumped. The flow of the Detroit River is given as 225 000 cu. ft. per second. From all available information as to the velocity of the water, it was found that it would reach Detroit in not less than 6 and would probably require not more than 10 days. From a careful study of the statistics of typhoid fever, as published by various Boards of Health, the conclusion was reached that the average period of incubation was 14 days. An examination of the statistics as to this disease showed that death, when it ensues, ordinarily occurs between the twenty-first and twenty-eighth days, which would give 25 days as an average. From April 16th, the day on which the first scow load of polluting material from the sewers of Port Huron was dumped into the St. Clair River, until June 5th, when there were four deaths from typhoid in Detroit, is 50 days. If 10 days are allowed for the water to flow, 14 days for the disease to incubate and 25 days to kill, the total is 49 days.

The investigation was carried on throughout the summer and the curves of secondary and tertiary infection were studied, and finally, about seven weeks after cold weather set in and dredging operations were suspended, the typhoid ceased. Next year the typhoid appeared again after the dredging had commenced, and when it stopped the disease disappeared. These conditions have followed one another since that time, and investigations have shown that they prevailed in previous years. Going as far back as 1886, it was found that typhoid appeared during those years when dredging operations disturbed the channel of the St. Clair River or Lake St. Clair above the intake of the Detroit Water-Works, even as far away as Lake Huron; and the relationship could be traced accurately. In one or two years, when there has been no disturbance of this channel, Detroit has had a very low typhoid death rate.

Another interesting case occurred in 1897, which was a year of very low typhoid in Detroit, the rate being only 1.1 per 10 000 living. During that year there was only one group of cases which could be considered as epidemic, and that came in August. In two weeks there were seven or eight deaths. In looking for the cause it was found that the Government was dredging in the Clinton River. The work had been commenced some time before, and the dredge had been working gradually up the river, the material being dumped in Lake St. Clair, in what was supposed to be stagnant water, at a point about 16 miles from the water-works intake. At a certain date the dredge arrived at a bar in the river just below Mount Clemens, a town which, though not having a complete sewerage system, has a few sewers discharging into the Clinton River. A computation, similar to that previously described, showed that the group of cases in August was due to infection stirred up by these dredging operations.

These instances are given for the benefit of those who speak of streams which are not used as sources of water supply. There is no stream which is not a source of water supply for some person or community, and there is no danger of going too far in the restriction of the pollution of our streams. Mr. Williams.

PETER MILNE, Assoc. Am. Soc. C. E. — The American Water-Works Association has had the subject of stream pollution under consideration for the past eight years. Judging from correspondence with State Boards of Health and eminent medical authority, the speaker is led to believe that interstate legislation will not accomplish the object sought; and that it will be necessary to appeal to the Congress of the United States to seek the enactment of national laws to prohibit the pollution of water supplies. Such law, may of necessity guard riparian rights and also protect the health of the people using the water of streams liable to be contaminated by sewage of every character. Mr. Milne.

A special standing committee of the American Water-Works Association has nearly ready for publication a voluminous report upon this subject.

An expression of opinion upon this important question by this Society will carry weight and influence and be accorded the consideration due.

An example of the manner in which a community may recede from its position taken regarding sanitation of water supply, is afforded by the experience of the City of Brooklyn, N. Y. More than four years ago, two streams, which formed a part of the daily supply, were condemned by the Board of Health of the City as unfit for use, on account of pollution due to the sewage of the population dwelling upon the water sheds, which pollution had increased yearly in proportion to the increase of population. These stream supplies were abandoned. In the present emergency and extremity of the city, viz., a long drought, it is suggested to again use the water of these streams, provided it is filtered.

The question of filtration being one of degree, it is apparent that its range of effectiveness in application is contingent upon the measure of pollution to be met and treated. If pollution is permitted to increase in intensity as well as quantity, it is reasonable to assume that the cost of filtration would be prohibitory and compel the consideration of new sources of water supply remote from the influence of pollution.

If the Federal Government enacts laws which in effect restrict pollution and contamination of the waters which are used by the people, it must invoke the skill of the engineer to aid it, on the one hand, and wise legislation, on the other hand, in order that equitable relations may be established and that the rights of all interests may be safeguarded in the effort to prevent the defilement of water supplies.

Mr. Allen. KENNETH ALLEN, M. Am. Soc. C. E.—In studying questions relating to stream pollution it is natural to turn to England, where in official reports on the subject there is a storehouse of information which is not to be found elsewhere; although of recent years the investigations undertaken by several States—notably Massachusetts—and by several cities, have made available much information of the greatest value. Increasing density of population, the concentration of large industries, and a general awakening of the masses to the importance of their sanitary environment, all serve to direct attention to the matter of stream contamination.

Up to the present time, the most important legislation on the subject has been the Rivers Pollution Act of 1876, resulting from a report of the Second Rivers Pollution Commission on the Rivers Mersey and Ribble, in which the following requirements, especially regarding organic pollution, which had been recommended by Dr. Frankland, were adopted:

"We suggest that the following liquids be deemed polluting and inadmissible into any stream:

"(*a*) Any liquid containing in suspension more than 3 parts by weight of dry mineral matter, or 1 part by weight of dry organic matter, in 100 000 parts by weight of the liquid.

"(*b*) Any liquid containing, in solution, more than 2 parts by weight of organic carbon, or 0.3 part by weight of organic nitrogen in 100 000 parts by weight.

* * * * * * *

"(*f*) Any liquid which, after acidification with sulphuric acid, contains, in 100 000 parts by weight, more than 1 part by weight of free chlorine.

"(*g*) Any liquid which contains, in 100 000 parts by weight, more than 1 part by weight of sulphur, in the condition either of sulphuretted hydrogen, or of a soluble sulphuret."

Although adequate provision for the enforcement of this Act appears not to have been made, its requirements were probably as fair and sound as was possible with the scientific knowledge of that time.

In the same year, 1876, the Massachusetts State Board of Health presented, in its annual report, a number of recommendations regarding the discharge of crude sewage into streams; among them:

"That no city or town shall be allowed to discharge sewage into any water-course or pound without first purifying it according to the best process at present known, and which consists in irrigation; provided that this regulation do not apply to the discharge from sewers already built, unless water supplies be thereby polluted; and provided also that such intended discharge can be shown to be at such point or points that no nuisance will arise from it."

But they are careful to add, that:

"The Board feels that, in the present state of our knowledge, sweeping laws for the general and immediate purification of all our streams would be hardly justifiable, and that they are not called for by the present condition of our rivers."

In the 23 years which have elapsed since these views were officially expressed, there has been opened up a vast field of research in sanitary matters, through the science of bacteriology, and, as our knowledge of the life history of these minute organisms is as yet quite imperfect, we are not likely to arrive at the ideal treatment of the matter for some years to come. Mr. Allen.

The standard of purity for sewage effluents, as established by the Second Rivers Pollution Commission, is faulty in the following particulars:

In the first place, the nuisance caused by a foul effluent depends quite as much upon the character of the stream into which it flows as on the effluent itself. A foul effluent discharged into a deep, swift-running stream of well-oxygenated water may cause no offence, while an effluent of half the size and twice the purity of the first, meandering over mud flats to a sluggish brook or mill pond may prove highly objectionable.

It is, therefore, seen that, if a standard of purity for the effluent is to be fixed, it should be in some way correlated to the character of the stream itself, and this under its most unfavorable conditions, as at extreme low water, when covered with ice, or when, for some other reason, it is particularly foul or devoid of absorbed air.

The two following examples serve to illustrate the inconsistency in the standard alluded to.

At the Leeds Congress of the Sanitary Institute of Great Britain, in 1897, Professor A. Bostock Hill, wishing to accentuate the difficulty of attempting to fix a standard for sewage effluents, quoted a case:

"Where a tank effluent, without any preliminary treatment, gave better results than the Rivers Pollution Commission standard, and yet subsequently so seriously polluted a stream that an injunction was applied for and obtained."

On the other hand, Alderman J. F. Spence (Chairman, Tyne Port Sanitary Authority), said:

"Between Tynemouth and Newcastle-on-Tyne, a distance of ten miles by the river, there is as large a number of works discharging foul and poisonous effluents as probably on any other stream in the kingdom. But these are so largely diluted by the enormous influx of sea water twice in the 24 hours, the difference in time between high water at the bar and Newcastle Quay being only 17 minutes, that the late Dr. Frank Buckland, when inspector of salmon fisheries, quoted the Tyne as one of the best salmon rivers in the kingdom, the pollutions being so diluted that the fish passed there unscathed to their spawning grounds in the higher reaches of the river. It may therefore be said the pollutions cause no nuisance, as the tide clears them all away."

Again, the Rivers Pollution Commission standard fails to appreciate the potential capacity for further purification in the effluent itself. This important point has been brought into prominence by the difficulty in securing permission, under existing laws, to discharge

Mr. Allen. the effluents from certain bacterial treatment plants into neighboring streams, although it is generally admitted that no nuisance would result thereby. The outcome of this is that a Royal Commission has been appointed, and is now giving this matter careful study, which will no doubt end in placing before the world most valuable and complete information on this subject.

The final purification of sewage, which is due to the mineralization of its organic constituents, is effected by the action of minute organisms, either vegetable or animal, but generally by bacteria, and this is the case with crude sewage, the effluents from chemical precipitation works or from sewage farms, as well as from the septic tank or other of the methods known more specifically as bacteriolytic. To interrupt this action by the use of antiseptics simply postpones the inevitable and desired result, allowing organic matter to accumulate. Such interruption is, therefore, fundamentally wrong.

The process of mineralization may be roughly considered as taking place in two stages.

In the first stage, the solids are broken up and thrown into solution by liquefying bacteria, while the rearrangement of the molecules of all the organic matter renders it more susceptible to further change. This action is called bacteriolysis, and it takes place in a comparatively short space of time, the characteristic product being carbon-dioxide. Although usually occurring under anærobic conditions—when it is often accompanied by the evolution of sulphuretted hydrogen, marsh gas and other offensive or dangerous compounds, and sometimes, very likely, by the formation of these virulent poisons known as ptomaines —it is pointed out by Dr. Samuel Rideal as quite probable that it may result from the action of such ærobic liquefying bacteria as *B. Prodigiosus* and *B. Liquefaciens* under suitable conditions.

In the second stage, which is marked by oxidation of the nitrogen, and known as nitrification, the bacteria essential to the process must be supplied with oxygen, and it is characteristically an ærobic process.

It is true that in the formation first of nitrites and then of nitrates the oxygen required for the former may, in the absence of free oxygen, be wrenched from the nitrates or from other combinations of oxygen, leaving the final conversion into nitrates to be effected later.

Dr. Walter E. Adeney, of Dublin, has taken advantage of this action in devising the method of sewage and sludge purification known as the "Oxygen System," in which nitrification is effected by the application of nitrate of soda or permanganate of potash, substances which are capable of furnishing the oxygen required.

Nitrification is a comparatively slow process, and, while accomplished in an ideal manner by intermittent filtration, yet by reason of its slowness it may often be effectively carried on in streams by means of the oxygen dissolved therein. When, however, this element is not

absorbed from the atmosphere as rapidly as it is taken up by the organic matter undergoing purification, anærobic conditions prevail. which we call putrefaction, with their attendant evils. Mr. Allen.

It is clear, then, that the ability of the stream to oxidize the impurities in an effluent is a most important consideration, which has been quite ignored in the standard set by the Second Rivers Pollution Commission.

The determination of the organic nitrogen and carbon as proposed by Frankland has been found difficult in practice, and has been largely superseded by the less precise but more practicable determination of (or test by) albuminoid ammonia, as proposed by Wanklyn, or of the oxygen consumed, as proposed by Forschammer, or by both of these together.

Messrs. Pearmain and Moor have recently proposed the following standard, as adapted to present needs:

	Parts per 100 000.
Total suspended matter (not to exceed)	2.0
Albuminoid ammonia "	0.2
Oxygen absorbed "	0.714
Total hardness "	28.57
Nitrogen as nitrates (not to be less than)	0.714

They insist on the last requirement for the reason that, besides indicating the completion of the process, the nitrates also indicate a store of oxygen which may be drawn upon to oxidize the remaining impurities, insuring thereby a guarantee against putrefaction.

Dr. Rideal, to accomplish the same end, has suggested, as a safer test than any previously proposed, "the ratio of the oxidized nitrogen in the form of nitrate and nitrite to the 'oxygen consumed' figure," as the former represents the oxygen available and the latter the oxygen required to carry out the process of purification. He states as his belief that when the available oxygen is in excess,—

"Then under no subsequent conditions would a reversion of the sewage take place so as to cause a nuisance, and such a method of valuing the suitability of an effluent was independent of the size of the stream or the dissolved oxygen it contained. When, and in the majority of cases this was the case, the stream was many times the volume of the sewage and contained dissolved free oxygen, then this free oxygen was utilized instead of the oxygen of the oxidized nitrogen. Under these conditions it was not necessary for the oxidized nitrogen as compared to the 'oxygen consumed' figure to be so large."

If the character of a proposed effluent, as well as its probable dilution, in running water could be predicted with certainty, its liability to cause offense could be pretty well foretold. A rough but convenient limit was determined in this way by F. P. Stearns, M. Am. Soc. C. E., when Chief Engineer of the Massachusetts State Board of Health, substantially agreeing with a similar limit fixed by Rudolph

Mr. Allen. Hering, M. Am. Soc. C. E., with reference to Chicago's sewage discharge, in which he designates a stream flow of not less than 2½ cu. ft. per thousand population as sure to cause offence, and a flow of over 7 cu. ft. per thousand population as not likely to cause offence.

Such a rule will, of course, apply only when manufacturing wastes are negligible in quantity or character; but where questions involving the dilution of crude sewage are concerned, this method of predicating offensive conditions is likely to prove as convenient and satisfactory as any, considering the number of indeterminate factors involved.

Whether more definite tests, to cover various conditions, are practicable remains to be seen, but the more recent ones appear to be steps in the right direction. Possibly the future may discover some simple biological tests by noting the effect of polluted streams upon various kinds of fish or other organic life contained in them. In fact, in the elaborate series of experiments on sewage disposal which has been carried on for several years past at Leeds, one test of certain effluents was their capacity to support the life of carp, the effect on confervoid growths being also noted. It is known that different kinds of fish, such as salmon, bass, catfish and menhaden, require waters of different degrees of purity, and their survival would, to a certain extent, be a measure of the contamination of the water in which they were placed.

The relative efficiency of bacterial treatment, as compared with mechanical filtration and chemical precipitation, is well illustrated by the following results obtained at Leeds:

	Oxygen Absorbed in 4 Hours.		Albuminoid Ammonia.	
	Parts per 100 000.	Percentage purification.	Parts per 100 000.	Percentage purification.
Crude sewage	12.14	....	1.21	....
Filtration through paper	5.53	54	0.67	45
Lime precipitation	6.57	46	0.64	47
Bacterial action (Dibdin's filters)	0.91	92	0.125	90

A great economic advantage in biological treatment is the elimination of the vexed question of sludge disposal. It is true that sand and other grit still have to be removed, and that woody fibre requires time to become peptonized without clogging the filter, but the troublesome and offensive organic solids may be effectually disposed of, and this is a great step in advance.

Crude sewage contains several million bacteria per cubic centimeter —London sewage some 3½ million. After complete nitrification, these are reduced to an insignificant number; but before that time, although an effluent may contain sufficient oxygen to complete the process of purification, it may contain a very large number of these organisms.

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Now, while there is some evidence to indicate that pathogenic varieties will have been eliminated before arriving at this stage, yet with the present imperfect knowledge, and with the possibility of their subsequent introduction into an effluent yet containing nutrient material, it would probably be wise to fix a low limit to the bacteria per cubic centimeter permissible in a stream used for a water supply which has received the effluent from biological filters. In fact, it is a question whether the water of such a stream should not be filtered before distribution to a city, as there is always a chance that at times, through accident or excessive flow, the purification of the sewage may not be up to the established standard.

When the stream is not used for drinking purposes, the presence of of a large number of bacteria in the effluent is not necessarily an unfavorable indication.

It is believed that, for the present, legislation regarding stream contamination and sewage purification should be largely tentative, both because experience in these matters is as yet limited, and because it is rapidly increasing.

In dealing with a subject so complex and, at the same time, often involving the public health and the outlay of large sums of money—in the establishment of purification works on the one hand and the depreciation of riparian property on the other—each case of pollution should be considered by itself, by an authorized official, who should be a man of much tact and ability, as well as one having special knowledge and experience in his line. He should be guided by certain general laws regarding stream pollution, but should be given considerable discretionary power, and, to avoid troublesome changes of policy, it would probably be well to have him serve a long term of years. On the other hand, he should not be enabled to use his authority in too summary or arbitrary a way, to the disturbance of vested interests, when not absolutely necessary for the public good.

In order to make his supervision effective, the area under his control should be defined by the natural water-shed lines; that is, he should have authority over the entire drainage area of one or more streams, and be responsible for their condition.

In order to effectively control stream contamination, it is suggested that all streams may be placed in one of three general classes, viz.:

I. Streams far from populous centers and not likely to be used for manufacturing or domestic purposes.

II. Streams likely to be used for other than domestic purposes.

III. Streams likely to be used for domestic purposes.

Such a classification would not appear to be difficult or intricate, and laws regarding permissible pollution would apply, in general, to all streams of the class in question. Class I would require little or no legislation, while certain standards of purity for sewage effluents,

Mr. Allen. etc., would apply to Class II and others to Class III. Moreover, in the case of large rivers, like the Mississippi or Hudson, it would be quite possible to include the headwaters and upper reaches in Class III and leave the lower reaches in Class II. On the other hand, if certain populous centers toward the mouth of the stream should befoul its waters to the detriment of the fishing or other interests near by, it might be necessary to create a special district to which the restrictions for Class III would apply.

That the purification of the sewage of large cities by land disposal is feasible, is seen by such examples as Berlin and Paris.

Suitable land, however, is not always available, when chemical treatment followed by filtration through the natural soil, as at Birmingham and Manchester, or through prepared filter beds, as at Glasgow and Sheffield, may be resorted to. In the cases of London and Salford, the sewage is merely clarified by chemical precipitation, and the sludge is carried out to sea by steamers and dumped; and in such cases the effluent cannot be said to be truly purified. The previous examples, however, suffice to show that there is no inherent difficulty in purifying the sewage of a large city, even if irrigation cannot be availed of. It is merely a question of selecting the best and cheapest.

The modern bacteriolytic processes avoid the acquirement of extended areas for irrigation on the one hand and the question of sludge disposal on the other, and from them the ideal solution of the sewage disposal question of the future may be hoped to be realized. Experience has already been sufficient to obtain for these methods very high endorsement.

For instance, in testimony before a Local Government Board Inquiry at Exeter, regarding the septic tank treatment, Dr. Rideal gave as his opinion, that

"Wherever a sewage farm would work, this system would work, and anything which would prevent this system working would prevent a sewage farm working."

When asked whether it could be applied to very much larger towns, he replied "much larger." Dr. Dupré, also, declared that "no system of purification hitherto known had such a complete effect upon sewage."

It must be admitted, however, that up to the present time no great city uses any of these methods. The largest plant is at Sutton, England, where some 145 000 U. S. galls. per day are treated by the Dibdin system of double filtration.

The Sewerage Commission of the City of Baltimore, which for some time has had this very matter under consideration, while admitting the admirable results attained at Sutton, Exeter, and elsewhere, and after consultation with Mr. James Mansergh, regarding this particular point, has refrained from recommending any one of these sys-

tems for so large a city, involving, as it must, much that would be new when applied on the scale proposed. But this decision is qualified by the statement that Mr. Allen.

"The Commission fully recognizes that one or more of them may prove available for the treatment of the sewage of some of our outlying districts, and may, if future experience sustains the claims and anticipations of their advocates, yet become adequate to our service on a larger scale."

To sum up what has been said, it is believed:

(*a*) That stream contamination should be prohibited, and the purification of the sewage of cities be compulsory only:

(1) When it is offensively evident to the senses of smell or sight.

(2) When manufacturing, fishing, or other productive interests are likely to be seriously affected thereby.

(3) When the health of neighboring communities is jeopardized, either through the authorized use of the stream for domestic purposes, the probable contamination of areas devoted to the culture of shell-fish, or the probable formation of deposits upon shoals or shores.

(*b*) That the purification of the sewage of large cities is always feasible. It is always costly, but it is probable that the introduction of bacteriolytic methods of treatment will, in general, when more fully developed, tend to economy.

(*c*) That to make legislation effective:

(1) All streams should be classified according to their permissible contamination.

(2) A responsible officer should supervise and control all sources of pollution throughout the drainage area of every stream in which pollution is restricted by law.

(3) Where possible to formulate consistent regulations regarding the limits of contamination, this should be done; but that with regard to organize contamination, in the present imperfect but advancing state of our knowledge on this subject, the matter may best be left largely to the discretion of the official in charge of the particular drainage area in question.

James Owen, M. Am. Soc. C. E.—It has not been regarded as within the province of this Society to discuss legal questions, but recognizing legal enactments as the crystallization of public opinion on the subject legislated upon, the legal aspect of stream pollution is worthy of consideration. The radical, unqualified proposition that all pollution of streams should be summarily stopped seems to the speaker unwise. The question is a very live one in New Jersey, and the conditions in the northern part of the State furnish an object lesson. Mr. Owen.

Mr. Owen. The Passaic River was formerly known as "the beautiful Passaic," now it is called "the dirty Passaic." Along its banks is a population of about 400 000 persons, who, by the construction of sewers, have completed, within the past twenty years, a system of discharge which renders the river unfit for potable uses, or for any purposes for which water can be used. Until three years ago, Jersey City took its water from the lower Passaic. It was bad water, much contaminated and unfit to drink, and Jersey City vainly tried, for years, to obtain from the Courts means to restrain the towns above its intake from discharging their sewage into the stream.

Newark obtained a new water supply about ten years ago, and Jersey City got a new supply three years ago. Curiously enough, as soon as Jersey City stopped taking its drinking water from the river, there arose a popular demand that the pollution of the river should cease. Paterson, with 100 000 population, is discharging a large amount of sewage into the river, and Newark and the towns between Newark and Paterson wish defilement from this source prohibited. Newark also discharges an enormous amount of sewage into the river, and this drifts up with the tide for four or five miles above the city.

On the Upper Passaic is the small town of Summit which is discharging into the river, not crude, but filtered, sewage. The City of Paterson has brought suit against Summit to prevent its pollution of the stream. As a kind of cross-fire comes the proposition of the City of Paterson to enjoin the East Jersey Water Company from taking water from the river, on the plea that there will not be left in the river sufficient water to take the city's sewage down. From this it will be seen that the trend of public opinion in that locality is indefinite.

It cannot be too strongly emphasized that there should be no contamination of drinking waters or the sources of such waters; but the matter of the discharge of sewers into rivers used merely for navigation, or for drainage, should be considered as a question of dilution; and if, within the next few years a standard of permissible dilution, which would be free from effects deleterious to the community, can be established, the prime object of this inquiry will have been attained.

Mr. Darrach. Charles G. Darrach, M. Am. Soc. C. E.—The study of this problem for twenty-five years leads the speaker to agree with the conclusions so admirably expressed by Mr. McMath. The engineers of America are competent to solve this problem, and the speaker anticipates not only its solution but such a development in the future as will modify the water-supply question to the extent that if necessary a city will not be obliged to go beyond the purification of its own sewage to supply itself with water.

Mr. Harrison. E. W. Harrison, M. Am. Soc. C. E.—The subject of the right of cities on tidal streams to discharge their sewers therein has not been taken up in this discussion, but some recent events indicate a disposi-

tion to impose unnecessary and harmful restrictions on such com- Mr Harrison.
munities. When the City of Newark was founded, about 200 years ago, the site was chosen because it was on a tidal stream. It offered, by reason of its location, advantages for settlement, for manufactures and for commerce. This agitation for the purification of the Passaic River, of which Mr. Owen spoke, resulted in swinging the pendulum far from the stand taken by the courts ten years ago, for in March last the City of Newark was enjoined from draining about 130 acres of land into the Passaic at a point where the tide ebbed and flowed. The water at this point was salt, though not as salt as the sea, but both medical testimony and a bacteriological examination were submitted to show that the growth of bacteria was inhibited by reason of its saltness. The river was thus acting only as a carrier, conveying sewage to the sea without affording opportunity for the further growth of bacteria.

That agitation and decision are now bearing strange fruit. Other towns draining into estuaries of New York Bay are seeking places of deposit for their sewage, and they can keep their water supplies pure only by turning their sewage into the tidal streams. This decision has helped to create the opinion in northern New Jersey that no sewage should be emptied into any stream, tidal or otherwise, not even into the Kill von Kull, a tidal strait between Staten Island and New Jersey, passing an enormous quantity of water with a swift current.

The recognition of this argument as a governing principle would operate to deprive cities located on estuaries and tidal streams of one of their great advantages as manufacturing sites. If a manufacturing plant cannot be established along the Arthur Kills, or a large city grow up there with the right to discharge sewage into the salt water; if investors in such enterprises must contemplate an enormous bond issue to take care of sewage, the manufacturing plant or the city might as well be located up in the mountains. Engineers will discriminate between drainage into salt water and drainage into a fresh-water stream, and, while the speaker agrees with Mr. Owen that the pollution of streams now used or likely to be used for water supply should be prohibited, the same arguments do not apply in the case of tidal streams; otherwise sewage purification works would be required for cities like New York and Brooklyn.

P. A. Maignen, Esq.—In Paris the distribution of the water supply Mr. Maignen.
is double and of two qualities; raw river water for industrial purposes, street sprinkling, fire extinguishing and other base uses—sold at 12 cents per 1 000 galls.; and spring or assimilated (filtered) water for drinking and household purposes—sold at 24 cents per 1 000 galls.

The potable water distribution, being kept at a higher pressure than the industrial water, became largely used for hydraulic elevators and other purposes as motive power. In order to discourage this

Mr. Maignen. misuse of the pure water, the rate for such purposes was raised to 48 cents per 1 000 galls. This differential rate has prevented the abuse of the pure water, so that filtration plants have to be provided only for water legitimately used and well paid for.

Another reason for which French engineers advocate the double canalization system is to maintain an even pressure in the pure water distribution, so as to reach the highest points of the cities at all times. It is customary, in many towns, to water the streets and flush the sewers, for several hours every day. With a single canalization the upper parts of the city would be deprived of water by the simultaneous opening of the low-town hydrants; this trouble does not occur with a double canalization.

Filtration, as an art, is still in its infancy. It is a mistake to ask too much of a single filtering operation. The work should be divided. Something has already been accomplished in this direction by the use of storage reservoirs, but unless they are, as in London, large enough to render pumping during storms unnecessary, they are only extensions of the river, and of little use. If storage reservoirs are wanted in case of accidents to the pumps, by all means build them, but, in the speaker's opinion, clarification can be obtained equally well by quick upward filtration through coarse sand or other coarse material—the choice varying according to the particular quality of the water to be dealt with—and at less expense.

Sand filtration, as carried on in Europe, with sundry improvements, could be applied as a secondary operation. And it is possible to find, in the large industries, such as sugar and chemical works, breweries, wine producers and the like, that have recourse to filtration, some ideas which may be of service in carrying out a third and final operation in the purification of water for city supplies.

Mr. Tribus. L. L. Tribus, M. Am. Soc. C. E. (by letter).—For twenty years past the providing of suitable water supplies for thickly settled districts has been receiving more and more intelligent consideration; the relation of physical health to the wealth and efficiency of a community has been more carefully studied from the economic as well as the æsthetic standpoint; and to the ends at issue have been brought the brains, energies and conscientious efforts of engineers, chemists and even of municipal officials themselves, who have been popularly classed as men devoid of other than political instincts.

To a more limited degree, the disposal of the wastes of civilized life, passing off as sewage, has received careful attention, and is beginning to be treated as it deserves, as one of the most important subjects for the engineer to consider. It is not alone necessary to provide human beings with pure food and drink; it is equally necessary to remove the refuse after the valuable parts are used.

The questions presented for these informal discussions, relating to

Stream Contamination by Sewage, Purification of Sewage, and Filtration of Water, are so correlated as to make their separation here scarcely wise, although certain features of each are distinct. Mr. Tribus.

By far the larger number of cities (the term "cities" including all communities making use of municipal systems of water supply or sewerage), take their supply from rivers or lakes, larger or smaller as the case may be; those using the various forms of driven and artesian wells not, as a rule, having the classes of waters that need artificial purification. This being the case, it is evident that the source of supply should be maintained in all possible purity, rather than that the water should be permitted to deteriorate, and then need to be purified by some form of filtration, and at great expense.

Consequently, defiling matter, as certainly raw sewage must be considered, should not be admitted to a lake or stream if there is any possibility of injuring the water supply of another place. There is no justice in permitting one community to save itself trouble and expense by entailing trouble and expense on another, or in event of that other not having sufficient enterprise to protect itself, and perhaps furnishing to it elements of disease and danger. Rights to the waters of a stream have caused litigation for centuries, but it is only in comparatively recent times that "use" of the stream has been considered as requiring, morally, if not always legally, passing it on in unimpaired purity.

Certain classes of sicknesses known as "Filth" diseases, of which typhoid is the most prominent, are usually attributed, and probably with justice, to the dissemination by the water supply (or milk, directly associated, too often, with it) of germs contributed by sewage to the sources of the water system. Dilution of sewage with water has been largely looked upon, even by engineers, as a means of purification, and to some extent it is, but dilution does not destroy germs; and disease germs in a large volume of water have just so much more chance of dodging the other forms of bacterial life which seek to destroy them.

Not many years ago, purification of sewage was not an unmixed success, chiefly because of unintelligent design and carelessness in the operation of plants, the method of Nature's action not being entirely understood, or, if so, then ignored; the germ theory had scarcely been promulgated, and the existence of bacteria of different kinds, which, like people, preyed on each other, was unknown.

It would be a difficult matter to estimate in dollars and cents the losses of property, the destruction of valuable material, and the injury to lives, directly traceable to the emptying of crude sewage into streams; yet few will deny that the aggregate sum would be vast. The driving away or destruction of fish life, removing thereby a very valuable article of food; the depreciated value of property adjoining such sewage contaminated streams, and the actual deaths of hundreds or

Mr. Tribus. thousands of human beings, from the use of polluted waters, sufficiently indicate the conditions at stake. The latter topic has been so admirably covered in recent papers on typhoid and allied diseases, with their relationship to water supplies, that merely speaking of the remarkable reduction in death rate, noticeable after the introduction of pure supplies, is all that is necessary here.

The New York State Board of Health, in its policy of saving streams and lakes from further contamination, practically compelling cities to treat their sewage, seems to the writer to have taken the only proper action that justice and right can countenance; yet there are instances where such exclusion would appear an unnecessary precaution, for the rights of no one would seem to be injured by "direct" disposal.

It is a question, therefore, that had best be left to the intelligent and more enlightened sentiment and judgment of communities directly concerned, trusting to fully sufficient existing laws to prevent undue trespass on the rights of others. The writer believes, however, that the proportion of sewerage systems to be constructed in the future, without adequate disposal and purification plants, will be very small; and that a large number of the systems now in use will ultimately require treatment.

The same discussion applies very largely to the second question—"Should the Purification of the Sewage of Cities be compulsory?" Present laws or court decisions cover fully all cases where non-treated sewage becomes a nuisance to residents of the city itself or to those of other places. Unfortunately, enforcement of laws seems to be a very difficult matter, and there is no reason to suppose that any new laws, even apparently more binding, would work to any greater advantage.

As far as applicability to large cities is concerned, new questions always arise. In almost all instances, large cities have grown up on the shores of tidal harbors, great lakes or large rivers, so that sewage flow bears usually so small a proportion to the volume of moving water, fresh or salt, that the dilution dissipates most of the cause of offence, though in the course of years, unmistakable and permanent indications of defilement can be noticed near the outlets of the sewers.

It is necessary to note clearly, what purification of sewage actually means. It is the total resolution into harmless inorganic elements, of all defiling or organic matters—a complete chemical combustion or oxidation. In this process, bacteria, abounding in the sewage itself, have a great deal to do; in the opinions of some, perhaps more than any other agent.

A direct answer as to its feasibility for large cities is impossible; each city must be judged by its own conditions. New York, Philadelphia and Chicago would illustrate the impossible, or at least, impracticable. Paris has, in a large measure, successfully embraced purification. London aims to make the effluent inoffensive, but it is

far from being purified; its chemical treatment and subsidence being only an arresting of Nature's processes, and not a completion of them. To a thorough understanding of this last subject must those come who have the handling of sewage problems. Nature cannot be hurried, though she enjoys being assisted. We must conform to her laws or failure will ensue. Mr. Tribus.

It is possible to purify sewage, and it is practicable, but it takes land in large areas to do it economically, and land properly selected and prepared, with the utmost care in its use, besides; or it takes water in such large volume that the same processes of nitrification and oxidation can be carried on without regard to time, and be conveyed away by currents or streams, so as to be inoffensive during the operations. The methods of treatment need not enter into this discussion; but it must be borne in mind that, unless local conditions absolutely prevent, some form of land treatment, without the use of chemicals, comes nearest to Nature's own most efficient process, and consequently is most likely to prove successful, even under severe climatic conditions.

Closely allied to the purification of sewage, and even more important to human life, is the providing of pure water supplies. Mountain lakes and brooks in uninhabited or sparsely settled districts can usually be taken as of sufficient purity, if the water be intelligently gathered and cared for.

The trend, however, of growth of population, is in the grouping of communities along the courses of the larger streams and on the shores of lakes. This entails more care in the selection of intake points, and is gradually tending to make undesirable the use of neighboring waters for public supplies without treatment before use. On the other hand, the smaller streams and lakes, not subject to pollution, do not furnish sufficient quantities of water for the larger cities, even when storm-waters, in addition to normal flow, are carefully conserved. It is not always feasible to secure ground water or rock well supplies, nor are such waters always safe or desirable themselves, so that recourse must be had in most cases to surface sources. Most persons will grant that a surface supply, from unimpeachable collecting grounds and stored in well-kept, clean reservoirs, is all that can well be desired, but in the more closely settled States, each year, such a condition is becoming much harder to find. Then, again, in the use of surface waters in their natural state there is always somewhat of an uncertainty, for, in spite of the most careful supervision of the water-shed, unless it be absolutely uninhabited, contaminating elements are always likely to appear.

The writer is not quite ready to insist that all public supplies be filtered, for here, as in sewerage questions, situation has a great deal to do with the method proper for adoption; but certainly, intelligent filtration never harms a water, while its lack may entail danger. Fil-

Mr. Tribus. tration, in some form or other, undoubtedly began with man's use of water; its adoption, on strictly scientific lines for public use, is a development of very recent years. We may neglect consideration of the small household filters, the thousand and one devices that improve or injure the water according to the care in their use, and take up the broader field of filtration of municipal supplies as a whole, by the two general systems which may be classed as "Chemical" and "Natural."

It is unnecessary here to discuss the comparative merits of either. It is sufficient to say that both have full fields for operation, the main point being that it is fully proven that impure waters can be filtered and rendered safe and desirable for domestic use. At the present day practically everything is measured by its value in dollars and cents, a materialistic theory which we have to recognize even while combating it as a principle, so that the *pro* and *con* of filtration for any particular city seems to be: Does the advantage of a purer water, in decreased death rate of population, figuring life at a minimum value, or a gain to the community of the work of so many persons annually, equal the operation, interest on cost and appropriations for a sinking fund for a filtration plant? Of course, taxes and politics, and various other, often conflicting, interests are also considered; though impaired health, more or less discomfort, and offensiveness to eye and nose, seem to have but little weight. Usually the activity of a violently deadly epidemic is necessary to impress the weight of the argument.

The writer is amply convinced that the coming solution of the public water question, considered as a whole, is in "purification," implying rather more than the narrower term "filtration," though that, in its growing ordinary acceptation, includes the very complex operations of Nature that result in thorough purification, hence safety for the health of, and satisfaction in other uses by, the public at large.

Mr. Ricketts. Palmer C. Ricketts, M. Am. Soc. C. E.—(by letter).—The great value and advantage of maintaining, if it were possible, all streams, large and small, in the purity they exhibited before there were settlements upon their banks is self-evident. The fact, however, remains that, to a greater or less extent, streams always have been and always will be natural sewers. They will be as long as water keeps flowing from the land into them and carrying with it impurities dangerous to health. Those flowing in thickly populated valleys certainly will be, even if the ordinary sewage of the cities on their banks be purified before being discharged into them. The fæcal matter of one inmate of one farm house may cause more havoc than a year's sewage flow from a large town.

As an example, consider the Mohawk River Valley in New York; narrow, with a long line of villages, towns and cities along its banks, and elsewhere quite thickly settled. If no sewage from towns and cities were permitted to be discharged into it the water would, never-

theless, be unsafe for drinking purposes, if not filtered. This river discharges into the Hudson River below the intake of the Troy water-works. Below Troy, the cities of Albany, Hudson and Poughkeepsie filter the water pumped from the Hudson River. At least the last two have filtered it for years and the first will shortly have its extensive plant in operation. Under these circumstances it would be extremely difficult to force the great population of the towns and cities in the Mohawk Valley to purify their sewage before discharging it into the river, in order that its water might be more pure, but not pure enough to be drunk with safety. Mr. Ricketts.

In the "Report on the Chicago Drainage Canal by the Commission appointed by the Mayor of St. Louis, Mo., 1899," may be found the paragraph:

"By general law of the United States all cities must, sooner or later, be prevented from polluting the natural water courses of the country with sewage."

However much jurisdiction the United States may have in the Chicago Drainage Canal question, it would have to be considerably stretched to cover the case of a non-navigable stream, like the Mohawk, lying wholly within a single State and discharging into a river lying, as far as that part of it valuable for water-supply purposes is concerned, in the same State. And being thus practically a State question, how much chance is there that this large part of the population can, by act of Legislature, be put to great expense without proof that it is necessary.

When the writer became engineer of the Public Improvements Commission of the City of Troy, N. Y., and had, in consequence, to design a complete system of sewerage for the city, he was asked whether it would not be advisable to build an intercepting sewer along the river and purify the sewage of the whole city at the south end before discharging it into the Hudson River. This would have entailed pumping 9 000 000 galls. a day, exclusive of any rainfall, to a height of about 200 ft., purifying it, and discharging the effluent into the Hudson about 5 miles below the mouth of the sewage-laden Mohawk River. Twenty-five years ago the City of Troy began to pump most of its water supply from the Hudson above the mouth of the Mohawk, where the water was quite pure. Instead of doing this it could have obtained a bountiful supply by a gravity system at a reasonable cost. In the meantime the river has become more contaminated above the intake, by the sewage of towns which have since grown larger, and the quite pure water supply still exists in the highlands back of the city. Under these circumstances could Troy, in equity, compel the towns above to purify their sewage?

The writer has thus taken up the conditions existing in one neighborhood to show that no radical general rule can reasonably be made

Mr. Ricketts. regarding sewage purification. General laws, absolutely prohibiting the discharge of unpurified sewage into streams, would in many cases, be destructive and unjust; and, in the design of a system of sewerage, it is well to remember that it is easy to put a community to unnecessary expense to meet possibilities which never materialize, and sometimes to prevent the construction of much-needed works by prohibitive and unnecessary conditions.

On the other hand, it is not necessary to state that many cases exist where the purification of sewage before its discharge into streams is advisable and necessary. This, of course, is particularly the case where the streams are small and the country thickly settled. Those in the neighborhood of Boston afford a good example. It also seems evident that even if a stream is not to be used for a water supply it should not be permitted to become a nuisance because of sewage discharged into it.

As to the pure water question for cities, the writer believes filtration to be its coming solution. More or less pure water may be used without filtering if it comes from a source uncontaminated by the sewage of villages or cities, but the only way to be sure that it shall be reasonably pure is to filter it.

Mr. Haring. James Smith Haring, M. Am. Soc. C. E. (by letter).—There is a phase of the discharge of raw sewage into rivers and streams which was not mentioned in the oral discussion of this subject, that of the minimum flow, in either rivers or streams.

This is a matter of great importance in considering the question of permitting untreated sewage to enter sources of water supply, not only as regards the effect upon the purity of the supply for other localities, but also the relation which the amount of sewage passing into the stream has to the quantity of water flowing therein, to prevent the nuisance of the collection of undiluted sewage in the immediate vicinity.

Most western rivers are subject to large variations in both depth of flow and area covered during the periods of maximum and minimum seasons. Although the volume of water is calculated from the total quantity passing in the channel, yet it may be so divided and diverted as to leave the conditions attending the dilution of sewage quite at variance with the permissible allowance. Especially is this true of rivers having very rocky channels, in which the flow is concentrated at certain points into small cross-sections; while, at other places, the water is spread out over a very shallow but wide and stony bed and almost disappears. This is also true of a river of sandbars and marshes. The collection of sewage in beds of this character certainly emphasizes the necessity of compelling its treatment before discharging it into streams used for water supplies. It seems quite unnecessary to require any treatment of sewage before disposing of it in tidal

salt water bodies. Cases might arise where sewage might be caught on a flood tide and carried back upon the shores and cause a nuisance, but the conditions would not be general, and the writer does not think that in the discussion of this subject such a contingency was anticipated. Mr. Haring.

The only way to make the purification of water supplies effective is to first prevent, by Federal laws, the pollution of rivers and streams. Too many of our rivers are in the jurisdiction of more than one State to make the State control of these matters effective, and the personal feelings of lawmakers are too largely acted upon by the fear of giving offense to some person or interest in their constituency to make a success of individual State legislation. Then, too, individual cities and towns will procrastinate to a degree almost, if not quite, criminal, rather than press on to a reform, or even to provide the means necessary to remedy or prevent the pollution of streams, because of the attending expense; and though conscious and confessing the, shall we call it, crime, permit it to go on. Only a general law, irrespective of State or city, can prevent effectually what will be in a few years, if it is not in some places already, an evil of great magnitude.

Filtration is destined to be the important agent to prevent wholesale contagion from water-borne diseases, even though it be compulsory by law to treat sewage before permitting it to flow into streams, and even though the effluent be purified "to a drinking-water standard," as one business concern engaged in that work now claims to do (though the standard may need qualification). The rapid development in the education of the people demands the filter as a guaranty of the best that can be had in water supply. Though sewage may be treated and made inoffensive by the expenditure of money, yet water supplies cannot be manufactured, and the daily increasing news of water famine all over the country admonishes us of such conditions.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.--This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

FRICTION COEFFICIENT FOR RIVETED STEEL PIPE.

An Informal Discussion at the Annual Convention, June 28th, 1899.*

SUBJECT FOR DISCUSSION.

"What is the Proper Friction Coefficient for use in the design of Riveted Steel Pipe?"

By A. McL. HAWKS, Jun. Am. Soc. C. E.

Mr. Hawks. A. Mc. L. HAWKS, Jun. Am. Soc. C. E. (by letter).—The writer has tested the flow in a 14-in. riveted steel pipe after it had been in service over three years, and again nearly three years later. As these measurements were carefully checked by volumetric determinations after weir flows were measured; as they check very closely with one another, and as they were made upon the main inflow of a system in the ordinary everyday uses, to which such a main is applied, they will be a guide to designing similar pipes for such purposes.

This main is made of soft steel, is 14 ins. clear diameter of the smaller ring, and has double-riveted longitudinal seams and single-riveted circumferential seams. The double riveting is spaced $\frac{4}{64}\frac{5}{4}$ in. between rows, $\frac{5}{64}\frac{4}{4}$ in. to $1\frac{3}{64}$ ins. between heads, and staggered. The circumferential rivets are $\frac{4}{64}\frac{5}{4}$ in. from center to center upon the exterior

* The discussion of this subject, for which no formal paper was presented, is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to September 23d, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

surface, or about $\frac{1}{2}$ in. from center to center on the interior surface, varying slightly with the thickness of the steel, the gauge of which runs from No. 9 to No. 12. All rivet heads are circular or "snap heads," and as the diameter of the rivets is $\frac{15}{64}$ to $\frac{21}{64}$ in., the heads form almost a complete ring upon the interior of the pipe, and practically reduce the clear opening about $\frac{1}{4}$ in. An attempt was made to compensate for this by making the interior rings a slight fraction of an inch greater than the size originally planned. As the outer rings were kept uniform, this increased size amounts to about $\frac{1}{8}$ in. in the lighter and $\frac{1}{16}$ in. in the heavier steel. Mr. Hawks.

Seven rings of steel plate (four of the greater and three of the lesser diameter) formed a 24-ft. length of pipe as made up in the shop. One end was reinforced with a 5-in. welded steel ring of 14 ins. clear diameter, the end lengths of the pipe being of the greater diameter and the end longitudinal rivets being practically countersunk to admit this reinforcer. In making up the pipe lengths the longitudinal seams were "in line" in alternate rings and "quartering" in adjacent rings.

All lengths were thoroughly cleaned and dipped in a hot asphaltum bath, being left a sufficient length of time for the metal to attain the temperature of the bath. All lengths were inspected and tested previous to leaving the shop. That this work was thoroughly well done is shown by the fact that some extra lengths and extra rings were left exposed to the weather along the line of the work, and the coating of these was in fairly good condition after more than six years.

In laying the pipe in the trench the longitudinal seams were placed so as to form an angle of 45° with the vertical plane; the reinforcer, which projected half its width from the end of the pipe, was forced into the next length until the exterior rings were in contact. An exterior welded ring $\frac{3}{8}$ in. thick and 5 ins. wide was drawn over the joint and an ordinary lead joint was poured and caulked.

As will be seen from the above, the obstructions to flow are: 1st, the circle of rivet heads of the transverse joints, about $3\frac{1}{2}$ ft. apart; 2d, the lap of the rings; 3d, the reinforcing rings, 24 ft. apart; and 4th, the double line of rivet heads making the longitudinal seams.

The total length of this pipe line is 70 700 ft., leading from an iron screen chamber, near the inlet, to the distributing reservoir. The profile of the pipe line is fairly smooth, there being four principal summits (two approaching the hydraulic grade line) and five principal depressions (two approaching zero of the datum plane); the remainder is chiefly along benches with easy grades. Automatic air valves are provided for all summits and blow-off gates for all depressions. The alignment is very easy, no quick bends occurring in the whole length. In two places where 90° curves are required they are made with six 15° cast-iron pipes. Therefore, no allowance for friction in bends is necessary in the calculations.

Mr. Hawks. This work was completed in the fall of 1892. In April, 1896, the writer tested it, and the following are his notes:

April 2d.—Visited intake and headworks. At 1.30 P. M. measured depth of water in screen chamber, which was 7.98 ft., making elevation of surface of water 429.15 above datum.

April 3d. Weir test.—The weir was a square notch, 15 ins. wide, cut in a sheet of No. 8 steel, set in the overflow chamber of the reservoir. All the conditions surrounding it were favorable to using the Francis formula, with end contractions: Discharge $3.366 \times$ (length of overfall$-\frac{1}{5}\ h$) $\times \sqrt{h^3}$, and this was used in calculating the discharge.

As the total outflow was only 1 cu. ft. per second and the area of the approach channel was 2.8 ft. $\times$ 3.1 ft., or nearly 9 sq. ft., the velocity of approach was disregarded.

Reading on gauge at	8	P. M.	4	ins.
"	" "	10	"	$4\frac{5}{8}$ "
"	" "	10.15	"	$4\frac{5}{8}+$ "

The weir was then closed for 15 minutes to allow the reservoir to fill to a higher level. It was reopened at 10.30 P. M., when the reading on the gauge was $4\frac{7}{8}$ ins. It gradually fell until at the close of the test it read $4\frac{5}{8}+$ ins., and remained stationary at that figure for 30 minutes.

Elevation of surface water = 396.95 ft. above datum.

April 4th. Volumetric Test.—The supply to the city was cut off at 7.20 P. M.

Gauge reading at	7.30	P. M.	11 ft.	$5\frac{5}{8}$	ins.
"	" 7.45	"	11 "	6 "	$\frac{3}{8}$-in. rise.
"	" 8	"	11 "	$6\frac{3}{8}$ "	$\frac{3}{8}$ "
"	" 8.15	"	11 "	$6\frac{7}{8}$ "	$\frac{1}{8}$ "
"	" 8.30	"	11 "	$7\frac{1}{4}$ "	$\frac{3}{8}$ "
"	" 8.45	"	11 "	$7\frac{3}{4}$ "	$\frac{1}{8}$ "
"	" 9	"	11 "	$8\frac{1}{8}$ "	$\frac{3}{8}$ "
"	" 9.15	"	11 "	$8\frac{5}{8}$ "	$\frac{1}{8}$ "
"	" 9.30	"	11 "	$9\frac{1}{8}$ "	$\frac{1}{8}$ "
"	" 9.45	"	11 "	$9\frac{1}{2}$ "	$\frac{3}{8}$ "
"	" 10	"	11 "	$9\frac{7}{8}$ "	$\frac{3}{8}$ "

The first and last readings were taken most carefully. Intermediates were taken to roughly check the results.

The total rise was 0.35 ft. in $2\frac{1}{2}$ hours.

After this test was completed the water was cut off from the reservoir, and it was left to stand full until morning. As the gauge read precisely the same at the end of 12 hours it was evident that no leakage of moment had occurred during this test.

The flow over the weir was calculated to be 0.96 cu. ft. per second.

Mr. Hawks.

Discharge $= 3.366 \times (l - \frac{1}{5} h) \times \sqrt{h^3} = 3.366 \times (1.25 - 0.08) \times \sqrt{(3.9)^3} = 3.938 \times 0.244 = 0.96$ cu. ft. per second.

The rate of inflow was 0.99 cu. ft. per second. The average area of the reservoir surface at time of filling = 25 225 sq. ft.

Rise of 0.35 ft. over an area of 25 225 sq. ft. =	8 829 cu. ft.
Allowance for voids in paving.....	65 "
Total.....	8 894 cu. ft.

Total inflow = 8 894 cu. ft. in $2\frac{1}{2}$ hours (9 000 seconds), or at the rate of 0.99 cu. ft. per second.

This rate of discharge agrees nearly with the calculated discharge by Kutter's formula if the coefficient of friction $n = 0.013$ be used.

$$S = \frac{429.15 - 396.95}{70\,700} = \frac{32.2}{70\,700} = 0.000455.$$

$V = C\sqrt{R\,S} = 87\sqrt{0.292 \times 0.000455} = 87 \times 0.0115 = 1.0005$ ft. per second.

Discharge $= 1.069 \times 1.0005 = 1.07$ cu. ft. per second.

In February, 1899, the writer was called upon to measure again the flow through the same main, under a slightly greater head. A weir box 22 ins. wide, 20 ins. deep and 9 ft. 3 ins. long, was built at the inflow of the reservoir, having a 90° triangular notch of No. 20 sheet iron at the outflow end. Through this the water flowed 8.9 ins. deep. Using Thompson's formula for the discharge of triangular notches: The discharge $= 0.0051 \times \sqrt{h^5} = 0.0051 \times 237 = 1.209$ cu. ft. per second.

A volumetric test was made by allowing the water to flow freely into the reservoir for 4 hours 5 minutes. The average area of the water surface was 13 149 sq. ft., upon which the rise was 1.34 ft.

Rise of 1.34 ft. over an area of 13 149 sq. ft. =	17 620 cu. ft.
Allowance for voids in paving.....	80 "
Total.....	17 700 cu. ft.

Total inflow 17 700 cu. ft. in 4 hours 5 minutes (14 700 seconds), or at the rate of 1.204 cu. ft. per second.

Notes of Volumetric Test.—The supply to city was cut off at 8.30 P.M.

Gauge reading at	8.45 P. M....	2 ft.	$1\frac{1}{10}$ ins.	
" "	9 "	2 "	$2\frac{1}{4}$ "	1.15 ins. rise.
" "	9.30 "	2 "	$4\frac{1}{4}$ "	2 "
" "	10 "	2 "	$6\frac{1}{4}$ "	2 "
" "	10.30 "	2 "	$8\frac{1}{8}$ "	$1\frac{7}{8}$ "
" "	11 "	2 "	$10\frac{1}{8}$ "	2 "
" "	11.30 "	3 "	$0\frac{1}{8}$ "	2 "
" "	12 M.	3 "	2 "	$1\frac{7}{8}$ "
" "	12.50 A. M...	3 "	$5\frac{2}{10}$ "	3.2 "

Mr. Hawks. At which time the reservoir was cut off for 12 hours. No leakage was observed.

The first and last readings were taken most carefully. Intermediates were taken as rough checks.

This rate of discharge nearly agrees with the calculated discharge by Kutter's formula if the coefficient of friction $n = 0.013$ be used.

$$S = \frac{430.3 - 389.0}{70\,700} = \frac{41.3}{70\,700} = 0.000584.$$

$$V = C\sqrt{R\,S} = 87\sqrt{292 \times 0.000584} = 87 \times 0.0132 = 1.148 \text{ ft. per second.}$$

Discharge $= 1.069 \times 1.148 = 1.227$ cu. ft. per second.

From these two tests the writer is led to believe that, for similar pipes under like heads, the coefficient of friction, $n = 0.013$, is approximately correct.

He also wishes to draw attention to the fact that there has been practically no decrease in the discharge of the main after it has been in use more than six years. From all the data he can gather, he is of the opinion that, for the first ten years of its life, such a main can be depended upon to give its full calculated discharge.

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ELECTRICITY VERSUS STEAM FOR BRANCH RAILROAD LINES.

An Informal Discussion at the Annual Convention, June 30th, 1899.*

SUBJECT FOR DISCUSSION.

"What are the economic conditions under which Electricity may be profitably substituted for Steam in the operation of Branch Railroad Lines, and what are the engineering requirements to be considered in such substitution?"

By Messrs. H. S. HAINES, H. G. PROUT, OBERLIN SMITH, W. B. REED, G. S. DAVISON, W. L. WEBB, J. JAMES R. CROES, W. J. BALDWIN, D. C. JACKSON, GUSTAV LINDENTHAL, EDWARD WEGMANN, ROBERT MOORE, C. W. BUCHHOLZ and A. P. DAVIS.

H. S. HAINES, M. Am. Soc. C. E.—As the speaker is responsible for the submission of this subject for discussion he will undertake to state the purpose he had in view in suggesting it. Mr. Haines.

Electricity has been gradually introduced as a motive power on railroads, first as a substitute for horse-car lines in cities; then into continually widening areas, with the extension of such lines into suburban districts; all the time offering more formidable competition to steam railroads, until it has become a question as to its substitution for steam on some of these very lines.

* The discussion of this subject, for which no formal paper was presented, is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. XXV, p. 71.)

Communications on this subject received prior to September 23d, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

Mr. Haines.

This problem has been attacked by one or two of our principal railroad systems in what may still be considered an experimental way, and it would be a matter of public utility if the results so far obtained should be gathered together and be made more accessible by publication in the *Transactions* of this Society.

The profitable substitution of electricity for steam in a railroad motor probably varies with its application, whether to the traffic on a strictly suburban branch, or to a branch in a rural or manufacturing district. There is also to be considered the advisability of its application alternately with steam on the same line. The conclusion seems to have been reached that this substitution can be more profitably made where it is desired to move single cars frequently rather than where it is required to move heavy trains at infrequent intervals.

The speaker thought of this subject because of a case that occurred in his own experience. In planning quite a long main line, he had occasion to provide for several branch lines to reach factory sites or manufacturing villages, at varying distances from the main line, where a mixed train making one or two round trips per day might meet all the requirements, at least for a year or two after the branch lines were opened. In one case it would be practicable to obtain current from a factory where it was generated by water power. In this way his mind was directed to the subject, but he was not sufficiently familiar with the economic conditions affecting the transmission of an electric current and its use as a motive power to determine the question satisfactorily to himself, and hence its reference to the Society for discussion.

The speaker is aware that there is a phase of the discussion of this subject which is more appropriate to a conference of expert electricians, but it is not that phase which he has presented here. It is distinctly as an engineering problem that he asks for a solution. There is an apparent disinclination on the part of civil engineers to consider, as appertaining to their domain, the application of electricity in connection with engineering works, in the same way that they accept and discuss questions involving the use of water or steam power.

As a help to himself, and to others similarly interested, the speaker would like to have a discussion of the conditions under which it would be advisable to operate by electricity a branch line to a steam railroad, whether in a suburban, a manufacturing, or a rural district; the conditions including the length of the branch, the character and frequency of the traffic, the generation of the electric current by water power or by steam, etc.

The subject should be considered, not only with reference to lines with a large traffic, but also as applied to branch lines operated only by one locomotive with a single train crew, making one or two round trips daily.

If these conditions can be determined with approximate accuracy, the field for the application of electricity will be accordingly extended, and perhaps it will thereby become more feasible to provide traffic facilities for communities, to which they cannot be profitably extended with steam as a motive power.* Mr. Haines.

HENRY G. PROUT, M. Am. Soc. C. E.—This question is pre-eminently one of traffic conditions rather than of strictly engineering conditions; although engineering covers traffic conditions. Under such requirements as Mr. Haines has suggested, *i. e.*, branch lines with infrequent trains, it would not be advisable to try to substitute electricity for steam locomotives; and it is quite likely that during the next few years light steam motors for such work will be developed. Mr. Prout.

The costly installation of the New York, New Haven and Hartford Railroad at Hartford and Berlin, Conn., will probably be found to be a very expensive and unremunerative experiment. It is said that about half of the machinery in the power house is lying idle and that one-car trains are run at intervals of twenty or twenty-five minutes. Under such conditions it is impossible to pay the interest on the cost of the installation. In other words, if the traffic is in units of considerable size at infrequent intervals the work can be done more cheaply by steam locomotives than by electricity.

A locomotive which will develop say 1 000 H.-P. will cost (we will assume) $10 000, and a power house and plant to develop 1 000 H.-P., will cost perhaps $80 000. Thus it will be seen that the interest on $70 000 has to be earned, or accounted for in some way by the superior economies of the electric installation.

It may be said that there will be a great saving in the fuel account, but careful analysis, made within the last three years, has shown that the fuel will cost as much per horse-power delivered at the rim of the driving wheel for electricity as for a steam locomotive. Although in a fixed plant cheaper fuel can be used, and with all the appliances of automatic stoking, etc., can be burned more cheaply than in the steam locomotive, yet the losses in efficiency of the generator, in transmission, and in the motor, are sufficient to overcome that saving, and by the time the power is delivered at the rail it will have cost as much in coal as by the locomotive. This seems to be a safe general proposition, although Mr. Baker, General Superintendent of the Manhattan Elevated Railroad, has lately said that many improvements have been made within the last year or two, and greater efficiency is obtained from the generators and motors. But even if a saving in coal of, say, 50% can be made, yet, after all, the fuel item in the operation of a railroad is comparatively small. Assuming the fuel element to be about 13% of the total operating expenses, then, even if the saving in this item is 50%, it is evident that little has been saved toward paying the interest on the expensive electric installation.

*For further remarks by Mr. Haines see p. 448.

Mr. Prout. If the traffic is such that its efficiency, or commercial development, requires a minute subdivision of the units, that is, a division into very small units running very frequently, in order to induce traffic to move, then, of course, electricity will win, not because it is cheaper than steam, but because it is adapted to such minute subdivision.

Mr. Smith. OBERLIN SMITH, M. Am. Soc. C. E.—For several years past the speaker has publicly advocated the use of small electric motors for driving machine tools and other independent machines. Although the idea at first appeared somewhat chimerical, individual motors are now extensively so used, the system having mostly passed beyond the experimental and transitional stages.

Railroad motive power is now in a state of transition. Within the last ten years the ordinary street trolley line has grown from a mere experiment into a great industry, which has not only performed the work of the horse-cars it displaced, but has created an enormous traveling habit. The time for the universal use of these motors on railroads has not yet arrived, but the speaker believes that it will come eventually, in consequence of discoveries which will cheapen the use of fuel and simplify electrical machinery; and also because such a method will result in greater convenience to travelers, by reason of more smoothness of motion, freedom from dust, smoke, cinders, etc., and facility for running single cars (or short trains) at much more frequent intervals than now. It therefore follows that the use of electricity on roads which are at present run by steam, will largely increase travel—and the habit of traveling. Such improvements make car riding a pleasure rather than the duty that it now often is.

The foregoing remarks apply to travel on surface roads, at speeds much the same as now practiced. The change in method has already begun, and its adoption by all railroads is simply a matter of relative cost—in construction, maintenance and operation.

It would seem probable, however, that in the next generation, if not in this, an entirely new type of high-speed railroad will be developed. Mechanically, the world is ready to begin it—financially, not yet, for its cost would be great. Upon some special type of track, confining the cars absolutely against derailment, a speed of 120 to 150 miles per hour, and possibly more can doubtless be obtained; and it would not seem to be a very wonderful feat to cover the 90 miles between New York and Philadelphia in half or three-quarters of an hour. There are no physical or mechanical obstacles which cannot be overcome with our present knowledge and some experimenting with details.

The danger of collision with other objects can be avoided by the use of an elevated roadway with absolutely no crossings, turn-tables or switches; and with the trains run upon a positive electric block system, in one direction on one track.

Mr. Smith.

The questions of derailment and collision being obliterated, the speed of trains will be limited only by air resistance and by the strength of the wheels to resist centrifugal force—if wheels are used. If the cars are made long, narrow, pointed, very smooth and properly jointed together when in trains of more than one, they can probably be run at high speed without encountering such great air resistance as would make the cost of power too great. Such cost, however, must not be based upon present train weights in relation to live load.

In an article in the *Forum*, for January, 1891, the speaker showed that the dead weight carried on American railroads, in many cases, amounted to 6 tons per passenger on trains of Pullman cars, and that a load of 1 or 2 tons per passenger is very common. From a mechanical standpoint this excessive weight is not necessary. The bicycle has shown that a person can be carried over a smooth dirt road with 20 lbs. of dead load, and on an iron pathway this can be further reduced; if riding on a tricycle covered from the weather, as in a car, this load might be 150 lbs., or about his own weight.

In this respect the bicycle has been an object lesson, and has shown that it is possible for a vehicle to be so delicately made, to have so much refinement in its mechanical design and composition, with ball-bearings, steel suspension-spokes, tubular frames, etc., that our old notions of wagons and cars appear absurd in comparison, especially when we carry 6 tons, or even 1 ton, of dead weight for each passenger.

It has been demonstrated that a heavy steam locomotive and train can travel upwards of 100 miles an hour, with all the disadvantages of a tremendous load not on the drivers, and a degree of uncertainty as to whether or not the train will stay on the rails. If, however, such a train could be made safe upon a surface road, at the speeds under consideration, and did it not cost too much for power, yet there are limitations of speed in respect to getting steam through the ports quickly enough and in working reciprocating members—as pistons, connecting-rods, etc. These limitations do not exist with rotating electric motors.

It would seem likely, therefore, that some of us may live to see some such railroading as has been indicated—a service safe, rapid and frequent, developed at first over short distances between very large cities. Should we succeed in adapting to the axles some form of suitable roller-bearings, we will save as much power as possible under the circumstances. Should we, further, be fortunate enough to find it practicable to equip our wheels with rubber tires, or some yet to be discovered equivalent, we then might get a smoothness and quietness of running which would indeed proclaim the advent of the ideal railroad.

One reason why electricity works better upon steep grades is that, with the best modern systems, every wheel in the train can be made a driving wheel. A train can be comparatively light, without the dead

Mr. Smith. weight of a locomotive, and yet have sufficient traction. With a steam train we not only have the cars very heavy, especially Pullman cars, but we have the big locomotive itself to drag up grades, with only the adhesion due to the weight of part of the locomotive, such as happens to be on the drivers. This is evidently very different from being able to have a very light train and making every wheel a driver, with every pound of the train and load contributing its share of adhesion.

Mr. Reed. W. B. Reed, Assoc. M. Am. Soc. C. E.—The discussion, up to this point, has referred mainly to passengers. The question of carrying freight in combination with passengers, upon electric lines, should also be discussed. Generally speaking, it has been found that for short distances passengers can be carried more comfortably and more cheaply by electricity than by steam, but in the carrying of freight little has been done by electricity as yet. That is a feature which will grow in the future.

When freight is to be carried over passenger lines by electricity it will have to be taken through city streets, or at least over highways, and the rails should be designed to meet the conditions of electric cars which have shallow flanges; freight cars with deeper flanges, and which vary more or less in gauge, requiring not only a deep groove but a wide flangeway; and the ordinary vehicle traffic.

Mr. Davison. G. S. Davison, M. Am. Soc. C. E.—About 8 or 9 years ago the speaker was in charge of the construction of a small branch line near Pittsburg which was required to connect a small manufacturing town with the Pittsburg and Lake Erie Railroad and the Pennsylvania lines. It was an independent corporation, controlled by manufacturers in the town, and it was equipped at first with steam power. The president of the company conceived the idea that with such cheap fuel as is to be had in that neighborhood, and in view of a proposed change in the future from steam to water power, it would be proper to operate this branch by electricity instead of steam. He experimented slowly, and equipped the line as a trolley road, but separated the passenger and freight traffic, and carried the former by electricity and the latter by steam locomotives.

The speaker does not know the results of the operations of this company, but judging from later occurrences he believes that the separation of the passenger and freight service was found to be a mistake. In this case the company had ideal conditions for cheap fuel, and had also its own electric power plant. It was not necessary to install a new plant, but simply to add to the old.

The road was afterward purchased by the Pittsburg and Lake Erie Railroad Company, and is now being operated as a branch of that road. The company, on acquiring this branch line studied its operation to determine whether it should be conducted as an electric line in part or as a whole, or whether it should be operated as a

steam road. As the electrical equipment was discarded and as the road is now being operated by steam, it may be assumed that electricity was found to be uneconomical. The causes which led to this change are not known to the speaker, but he suggests that possibly the trolley equipment might have been found very inconvenient when operating the road by steam as a freight line. The third-rail system may prove to be more practicable than the trolley in such cases, but the latter must be excluded on account of its inconvenience. Mr. Davison.

In the vicinity of Pittsburg there is a large mine which is operated for some 7 miles back from the point where the coal is brought to the tipple; the means of bringing the coal to the opening being virtually a small branch railroad. This branch passes through several tunnels and has been operated as a steam railroad for a number of years. At the present time the first 5 miles of the road is being operated as an electric line, using the trolley system, and on the remainder of the line, next to where the coal is brought out to the tipple, steam locomotives are used. The owner stated that the operation had been planned in that way, and that they had been justified in hauling the coal the first distance by electricity, because the use of steam locomotives in the mines was detrimental to the health of the men. But, taking full account of the cost of installation, power, etc., they had not found that it would be cheaper or better to continue the use of the trolley to the tipple.

The fuel of this power station is delivered directly from the mine into the boiler house, and yet, on the 5 miles operated by electricity, the saving on fuel is so small that the expense of an electric equipment for the remaining 2 miles would not be justified.

As the topic for discussion refers only to the operation of branch roads, in connection with a main line, Mr. Smith's proposition to attach a motor to every axle on the train would seem to be impracticable as it would involve the equipment of every car of the trunk line so that it could be used on the branch.

Walter L. Webb, Assoc. M. Am. Soc. C. E.—The relation of grades to the question of steam or electric operation has not yet been touched upon in this discussion. On a steam railroad, a grade of 2% is considered steep, while 4% marks almost the limit of working on such roads; yet on many of the electric railroads in the United States there are grades of 10% and even steeper. On steam roads which are steep in some places and flat in others there is a great waste of power. This occurs, not on the heavy grades where the engines are hard worked, but on the light grades where the great reserve power—at those points doing practically no work—has to be maintained for service on the heavy grades. On an electric road the amount of power used is almost proportional to the work to be done, and this is an important element in its economy. On a railroad in a mountainous region it Mr. Webb.

Mr. Webb. should be possible to adopt a cheaper form of construction than is necessary for a steam road with its requirements of low uniform grade, and to use freely very steep grades where desirable. Assuming a fair amount of traffic, so distributed as to keep the road in uniform operation, electricity will, in many cases, prove more desirable than steam locomotives, taking into consideration the interest on the cost of the construction and installation, and the expense of operation, and this will be true for these reasons alone, to say nothing of the possible economies consequent on a utilization of water-power.

Mr. Croes. J. James R. Croes, M. Am. Soc. C. E.—The economical generation of power, either steam or electric, implies a constant and continuous operation of the generating plant. When variable power is needed, the generating plant must be constantly operated at its maximum efficiency. When the work to be accomplished can be distributed over a continuous period of 10 to 24 hours, the power can be more economically generated by an indirect application through a stationary plant than by a direct application of the fuel to the motor, as in the steam locomotive. Where, however, the work involves a spasmodic application, occupying only a limited amount of time, at irregular periods during the day, the generation of power by an independent motor, capable of producing in itself the power needed, is more convenient and more economical than its production by a central plant.

Steam power at the present day can be generated spasmodically more efficiently than electric power. The requirements of railroad service on a line of limited traffic are spasmodic, reaching a maximum at certain hours; and until a storage battery has been devised in which the power produced continuously can be stored and used for spasmodic application, which application shall satisfy a maximum call greater than the average efficiency required of the generating plant, electricity cannot be applied more economically than steam. At present, it seems to the speaker that where the work is spasmodic, steam power is, and must be, more economical than electric power, for branch railroads.

Mr. Baldwin. William J. Baldwin, M. Am. Soc. C. E.—In many States the warming of cars is obligatory by law, and the cost of heating should therefore be taken into consideration in a comparison of railroad operation by steam and by electricity. About three years ago the speaker had occasion to test an electric heater. There was no unit of measurement to be determined, but the heater furnished was found, by comparison, to agree with 28 ft. of steam-heating surface. It was found that 28 ft. of steam-heating surface will condense about 7 lbs. of water in an hour, or about 0.25 H.-P. The electric heater, practically the equivalent of the heating surface mentioned, required between 6 and 7 electric H.-P. The result of this test indicates that warming by electric heaters requires 27 or 28 times the horse-power needed to warm by steam.

D. C. Jackson, M. Am. Soc. C. E.—There is no mystery about this matter, and Mr. Croes has well stated the case. It is more economical —not more efficient—to generate power for operating a railroad or a group of manufacturing establishments at a central station only when the power can be used continuously over long hours, for the reason that when the traffic on the railroad or the load on the motors at the manufacturing establishments is continuous, the demand on the central station for power is continuous, and the cost for interest and labor per horse-power hour developed by the station is brought to a reasonable figure. Where traffic requirements are intermittent, it is more economical to operate a railroad by steam locomotives. If a plant of 1 000 H.-P., installed to operate a branch railroad with freight traffic, is called on for its output for only 3 hours of the day, 24 hours of interest charges lie against the 3 hours of output. There are at the station, daily, possibly 10 hours of full labor and 14 hours of one-half or one-third labor, and if this is charged against the 3-hour output, it makes the cost of operating the station, per horse-power hour, enormous. Whether or not, electric power may be economical in various cases is, as Mr. Prout has said, a matter of the distribution of traffic. Mr. Jackson.

These remarks apply equally to electric power plants where the prime power is derived from steam engines or water wheels. The larger cost of development of a water-power plant will often cause an excess interest charge which is equal to the cost of fuel in a steam plant designed to do the same work, and the average small water-power plant does not embrace such economical advantages as are generally ascribed to it.

The case of a main line feeder, as put by Mr. Haines, is a special case under the general proposition, and under some circumstances it is possible that electric power can handle the traffic most economically. These conditions would assume a number of main-line feeders, located reasonably near a given point, so that the entire load of all these branches may be carried by a central station, using a fairly high transmitting pressure. The traffic should be broken into units somewhat smaller than would be operated ordinarily by steam power direct, and schedules of train movements should be so arranged that the station may be fairly well loaded during at least 12 hours. If the schedules are so arranged that once or twice during the day a single mixed train will be running simultaneously over each of these branches, the speaker doubts if electric power, from the point of view of dollars and cents, can ever compete with the steam locomotive.

Referring to the requirements for the electric heating of cars, some of the western electric street railways make provision for turning into the heaters during cold weather, as much power as is required to move the cars. In other words, in very cold weather, a station output of double that necessary to handle the cars is anticipated.

Mr. Lindenthal. GUSTAV LINDENTHAL, M. Am. Soc. C. E.—In discussing this question an important feature is the ability to store electric power. In recent years several electric power plants have been so constructed that, when running at a uniform rate, the power generated in excess of current demand could be stored up in secondary batteries acting as reservoirs, to be given out again whenever the demand for power exceeds the capacity of the plant.

A railroad train in descending a grade wastes power in grinding up brake shoes, and heretofore this power has not been stored in any form. It is possible to generate and store that power in the form of electricity, which can be used again in hauling trains or cars up an ascending grade, or for lighting or heating purposes. The speaker believes that the feature of storing electric power should have an important bearing in any comparison of the economies of locomotive and electric traction, particularly in mountainous countries where coal may be dear and water-power cheap.

Mr. Wegmann. EDWARD WEGMANN, M. Am. Soc. C. E.—The speaker objects to the statement that steeper grades can be used on electric tramways than on ordinary steam railroads. While it is undoubtedly true that steeper grades are used on trolley lines than on ordinary railroads, this difference should not be credited to electricity as against steam, but to the placing of a motor under each car. If a steam engine were applied to a car in this manner, the car could be driven up any of the grades to be found on trolley lines.

Before any trolley lines had been constructed the speaker had spent two or three years in assisting an inventor in developing and introducing a steam street car, which was run successfully in Brooklyn. This car, which was propelled by a small three-cylinder steam engine, placed between the axles and suspended from the car frame, was able to run up 10% grades.

While such a car might be used to advantage, where only a few trips had to be made each day, it had several disadvantages compared with trolley cars: 1st, The weight of the boiler was objectionable; 2d, A licensed engineer was required for each car. One of the great advantages of the trolley system is that men of ordinary intelligence can soon be taught to run the cars.

In the case of pulling a train by a special motor, the grade that could be overcome depended, as was well known, on the weight on the driving wheels, the rolling friction of the wheels, and the weight of the train. It made, therefore, no difference as regards the grades, whether steam, electricity, compressed air, or any other motive power were used to propel the motor.

Mr. Moore. ROBERT MOORE, M. Am. Soc. C. E.—Each particular case of this kind must be considered by itself. In the case indicated by Mr. Haines, in which the power is to be rented and paid for, presumably,

by a meter, the conditions of economy would be very different from those in which the railroad company would be required to install the plant, and bear the cost of the current whether it were being used or not. Therefore a proposition which would cover all cases might be very delusive. So far as a general proposition can now be made it must be substantially as indicated by Mr. Prout. Mr. Moore.

When the conditions are those of an urban traffic—a very large number of small trains—electricity has abundantly demonstrated its superiority over every other form of motive power; but when the conditions are very different from those of urban traffic—when the trains are infrequent or heavy—the speaker doubts its economy, and sees no indications that it is likely to be economical in the near future.

Regarding the superior economy claimed for the electric motor on heavy grades, the point cited by Mr. Wegmann is a sufficient answer. The power is developed only by the traction of the driver on the rail, and that is a function of its weight; thus, there can be no saving in that respect. But there is another point which should not be lightly regarded, the great danger, in operating on very steep grades, of the failure of brake power. Any such failure means disaster, as many who have had occasion to operate heavy grades well know.

In fact, if, with a possibility of doing otherwise, any one is tempted to use heavy grades, the best advice to follow is that of *Punch* to those about to be married—"Don't."

C. W. Buchholz, M. Am. Soc. C. E.—The facts are against the assertion that the grades on an electric road can be steeper than on a steam road. The power of a locomotive depends upon its adhesion to the rail, and it makes no difference whether the power is steam or electricity. Mr. Buchholz.

A. P. Davis, Assoc. M. Am. Soc. C. E.—This question is, without doubt, one of the kind of traffic, and to one more familiar with hydraulics than with railroad work, it appears to be a matter of saving, not 20, 40 or 50% of the fuel, but all of it. It has been pointed out that electric power makes its best showing in the mining regions, and one of its chief advantages seems to be that on steep grades it requires less dead weight than steam power, an important point in regions where steep grades are unavoidable. In mountainous regions the railroads, both main and branch lines, follow the streams, sometimes of heavy fall, where water power can be developed cheaply. In many cases in the West a water-power plant would not be chargeable with the total expense of the development, for whatever of diversion works and storage facilities are required are equally available as parts of an irrigation system which will receive the water after it has been used for power. Mr. Davis.

Two lines of railroad extend up the line of the Arkansas River,

Mr. Davis. cross the Rocky Mountains and follow the Grand River down on the other side. There, where coal is much more expensive than in the East, is an immense amount of power, constantly going to waste, which might be utilized by the electric system to move those frequent and heavy trains. Undoubtedly, the time is approaching when a great saving can be made in equipping with electric power, not only branch lines, but main roads as well, through mountainous regions, where water power is available. Improvements in the transmission of power are still possible, and the utilization of water-power in railroad operation is mainly a question of transmission. As transmission methods are improved, and as railroad traffic in such regions increases, the advantage of electric over steam power will be demonstrated with a definiteness to which the probable advance in the price of fuel will add its contribution.

Mr. Haines. H. S. HAINES, M. Am. Soc. C. E.—This question was suggested by the speaker because he considered it to be of practical moment, and because he desired to obtain from members of the Society an authentic expression of opinion as to how far, in the present state of the art, it was advisable to substitute electricity for steam as a motive power on branch lines of steam railroads.

If electricity is used on branch lines it must be used in connection with the vehicles in which passengers and freight are transported on the main lines. It is not only desirable to ascertain what can be done with electricity under this aspect of the case, but also what cannot be done. There is one general proposition that all will accept, viz., that in the present state of the art there is no doubt that electricity can be economically substituted for steam on a branch of a steam railroad where there are single units of transportation moving at frequent intervals, because the power can be used uniformly and continuously and because a large plant is not lying idle for the greater part of the time.

To state the cases in which electric transmission cannot be applied, a process of exclusion must be used; therefore, let it be said that it can be applied economically on branch lines for suburban purposes. If that is the case, it strengthens the steam railroads which have suburban branches competing with trolley lines. This seems to have been the idea which has prevailed in the minds of the management of some of the principal railroad systems operating out of large centers of population. The speaker recently had occasion to talk this matter over with some of the principal officials of the New York, New Haven and Hartford Railroad, who have probably had more experience in this matter than anyone else; and the impression received was that they are quite well satisfied with what they have done thus far in an experimental way, and they are extending the system. There is one point, however, which they have not yet determined, and that is, the con-

ditions under which the third rail should take the place of the trolley wire. Mr. Haines.

Admitting that electricity can be used economically on suburban lines the next question is: Can it be used on branch lines under other conditions? Judging from the preceding discussion the speaker believes that electric motors might be used to great advantage on branch lines in a manufacturing district, even for handling freight, if power can be obtained along those lines from manufacturers who have it to spare.

On a line running into a rural district it would seem hardly practicable to use electricity, unless, as Mr. Davis remarked, water-power could be obtained. That was one of the conditions in the case which was being considered by the speaker. The main line was for some miles parallel with a stream which it was intended to develop for manufacturing purposes; but the line ran along a ridge and the water-power was down in the valley. If branch lines could be run down into the valley the water-power there could probably be used more economically than steam.

The speaker need not discuss the question of heavy grades, as that has been covered so well by Mr. Buchholz. Unless by the use of electricity the traction can be made stronger than the mere adhesion due to the weight of the locomotive, that question need not be considered.

To sum up, it may be said that in the present state of the art it is only admissible to apply electricity on what may be called suburban passenger lines; except that it may be applied where power, for certain local reasons, may be obtained more cheaply than by means of the steam locomotive.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

GENERAL CRITERION FOR POSITION OF LOADS CAUSING MAXIMUM STRESS IN ANY MEMBER OF A BRIDGE TRUSS.

Discussion.*

By Mr. F. C. KUNZ.

Mr. Kunz. F. C. KUNZ, M. Am. Soc. C. E. (by letter).—The method of influence lines is not so often used as other methods, although it is the simplest way to determine stresses, even for uniform loading, in many cases, and particularly for web-members of all but Pratt trusses, bending moments of pins when simultaneous stresses have to be found, etc.

On page 78 the author gives a general rule for drawing influence lines. For use in special cases all similar methods derived from the theory of moments offer for web-members but little advantage, as the point of intersection of the top and bottom chords, lies, as a rule, beyond the drawing board. Of all the methods published on this subject, the writer gives in the following the simplest he has yet found. It was first published by Professor R. Land in Constantinople,† and applies to all cases of trusses with verticals and at least one horizontal chord.

For a diagonal $C_1 D$ rising to the left, Fig. 10 (and $C D_1$ rising to the right, Fig. 11):

Produce the top chord of the panel in question to its intersection A_1, (B_1), with the end vertical A, (B), and join A_1 and B, (B_1 and A); then the area $A_1 C_1 D B A_1$, (area $A C D_1 B_1 A$), is the influence area

* Continued from April, 1899, *Proceedings*. See March, 1899, *Proceedings* for Paper by L. M. Hoskins, Esq., on this subject.

† *Centralblatt der Bauverwaltung*, 1897.

Mr. Kunz.

when α = unit load. Its + (tension) and — (compression) value is found by simple inspection.

For a vertical D D_1, Fig. 10 (and Fig. 11):

Produce the top chord next to the vertical to its intersection A_1, (B_1'), as before; then the area A_1 D_1 E B A_1, (area A C D_1 B_1' A) is the influence area when h = unit load.

Professor Land derives these rules in a general way by means of cinematical principles. The writer gives, in the following, a proof, using the general rule on page 78.

Given four verticals F, A, C and D and a point B (Fig. 12). From B draw any two lines, join F_1 A_1, produce this to C_1, join C_1 D_1; then the locus of the point of intersection of C_1 D_1 and A_1 B is the same vertical z for any two lines drawn from B. Since (Fig. 13), $\frac{m_1}{m_2} = \frac{y_1}{y_2}$, and $y_1 = \frac{a\,m}{l_1}$, and $y_2 = \frac{a\,n_3}{l_2}$; hence $\frac{m_1}{m_2} =$

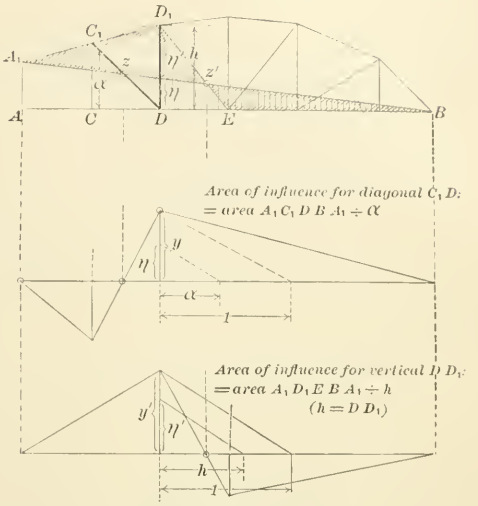

Fig. 10.

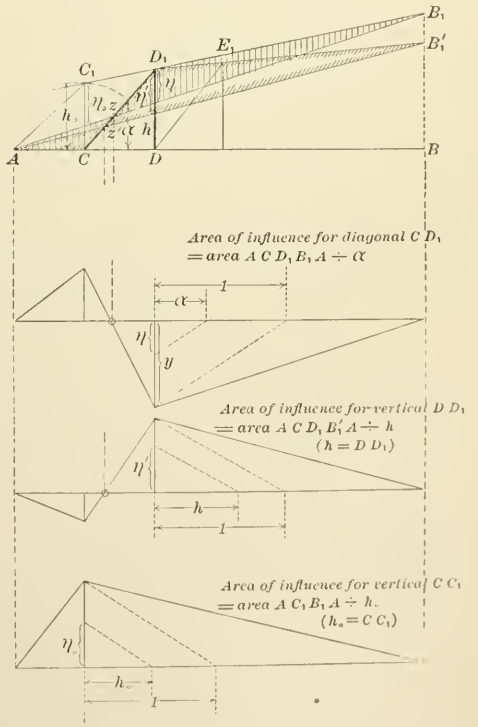
Area of influence for diagonal C D₁
= area A C D₁ B₁ A ÷ α
Area of influence for vertical D D₁
= area A C D₁ B₁' A ÷ h
(h = D D₁)
Area of influence for vertical C C₁
= area A C₁ B₁ A ÷ h₃
(h₃ = C C₁)

Fig. 11.

Mr. Kunz. $\frac{m\,l_2}{n_3 l_1}$, the position of the z-vertical depends only on the relative position of the verticals F, A, C and D and the point B.

In drawing the two lines from B through the skeleton of the truss in such a way that $B\ A$ corresponds to the one and $B\ A_1$ to the other, we see directly that as z_1 lies vertically above z it represents the zero point, and hence the area $A_1\ C_1\ D\ B\ A_1$ in the skeleton is the influence area of the stress in the diagonal $C_1\ D$.

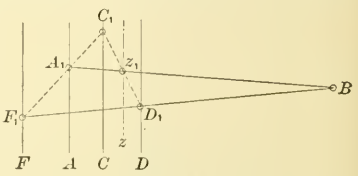

Fig. 12.

There remains only to find its scale. In other words, by what unit load α must η_1 be divided to give y_1, *i. e.*, $\frac{\eta_1}{\alpha} = y_1$ and $\alpha = \frac{\eta_1}{y_1}$.

$y_1 = \frac{a\,m}{l_1}$, substituting for α its value (see page 78) $\alpha = \frac{l\ l_2}{l\ t}$, we have $y_1 = \frac{l_2\ m}{l\ t}$. As $\eta_1 = h_1 - \frac{h_1 l_1}{n_1}\ \frac{n_4}{l}$ we get, after a short transformation, $\alpha = \frac{\eta_1}{y_1} = \frac{h_1\ t}{n_1}$. Further, $\triangle F\ G\ D \sim \triangle\ C\ C_1\ D$, hence $t = \frac{h\ n_2}{d}$, where d denotes $\overline{C_1\ D}$. Thus we get $\alpha = \frac{h_1^{\,2}\ n_2}{d\ n_1}$, and as $\frac{h_1\ n_2}{n_1} = h_2$, we get the unit load $\alpha = \frac{h_1\ h_2}{d}$, which can easily be scaled off in the skeleton. From the proportion $\frac{y_1}{1} = \frac{\eta_1}{\alpha}$ follows the construction of the influence line to any desired scale.

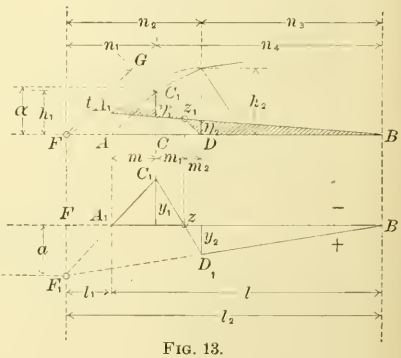

Fig. 13.

For the vertical $D\ D_1$ (Fig. 14) it follows that the point z_1 is the zero point of the influence area $A_1\ D_1\ E\ B\ A_1$. Using the notation in Fig. 14, we have again $y_1 = \frac{a\,m}{l_1}$, where $a = \frac{l_1\ l_2}{l\ t}$, and $t = l_1 + m$; $\eta_1 = h - \frac{h\ l_1}{l}\ \frac{n_4}{n_1}$.

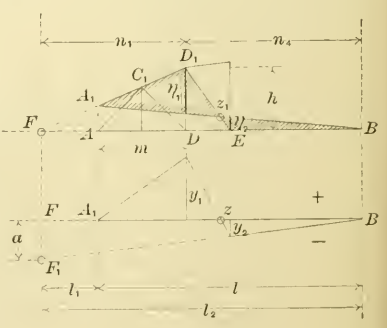

Fig. 14.

After a short transformation we get $\frac{\eta_1}{y_1} = h =$ unit load, and $\frac{y_1}{1} = \frac{\eta_1}{h}$. Mr. Kunz.

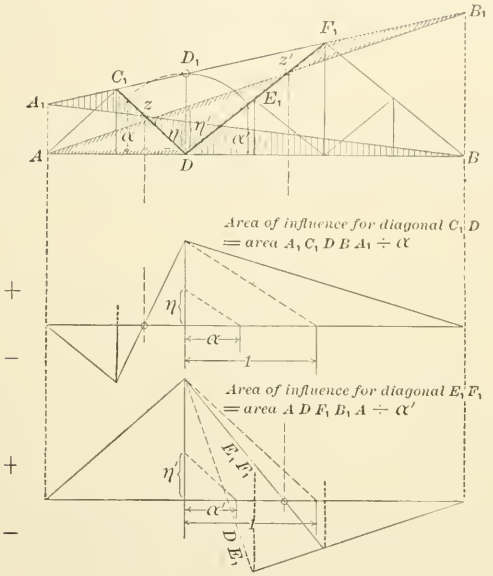

FIG. 15.

Fig. 15 shows the application of the above rule for the diagonals of a half truss of a drawbridge considered as a single span, which will be of particular use in determining the bending moment on the pin D for this condition.

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COMPARATIVE TESTS OF BITUMINOUS STEAM COALS.

Discussion.*

By Messrs. William H. Bryan and R. C. Carpenter.

Mr. Bryan. Mr. William H. Bryan (by letter).—This paper represents such an amount of experimental research, and the results have been compiled with such great labor and care, that considerable study is necessary to fit one to discuss it intelligently. This, taken with the author's high standing as an engineer, would, for the most part, tend to disarm criticism. The paper is unquestionably a most valuable addition to the data on the value of steam coals in everyday steam-boiler service.

The following points of criticism, however, would probably occur to the average engineer familiar with boiler trials:

First.—The starting and stopping of the tests do not seem to have been done by either the "standard" or the "alternate" method, recommended by the code of the American Society of Mechanical Engineers, which for many years has represented the best American practice. It is to be regretted that one or the other plan was not followed rigidly. The author states that the time of beginning and closing a test was when the coal charge on the grates was no longer able to maintain working steam pressure. This seems rather indefinite, and more likely to introduce error than either the "standard" or "alternate" method referred to. The writer's preference is for the "alternate" method, which he thinks would have given excellent results in the present case; and he doubts whether Mr. Hill's method

*Continued from May, 1899, *Proceedings*. See April, 1899, *Proceedings* for Paper by John W. Hill, M. Am. Soc. C. E., on the subject.

insures that the quantity and condition of the fuel on the grate are the same at the beginning and end of the test. The duration is, in all except six tests, exactly 16 hours, which leads to the suspicion that exactness as to the conditions might possibly have been sacrificed to uniformity of time. The duration of the tests, however, was sufficiently long to eliminate largely any errors which might have crept in, in starting and stopping. Mr. Bryan.

Second.—The use of meters for measuring the water seems unfortunate, in view of their known unreliability. The system of calibration was, of course, such as to reduce the possibility of error to a minimum, but the use of the ordinary weighing or measuring tanks would not have greatly increased the complication, and would have eliminated doubt.

Third.—Table No. 4 is a comparison of the boilers when clean and when dirty, and shows a fuel saving of $2\frac{1}{4}\%$ in favor of the clean boilers. The cleaning, however, seems to have been entirely internal, and no data are given as to the effect of soot on the external surfaces. The writer has found this to be a much more serious matter than scale, cutting down capacity and efficiency very rapidly. Many of the coals tested were very smoky. It is to be regretted that Mr. Hill did not at the same time make some record of the smokelessness of each of these coals, as they no doubt differed greatly in this characteristic. The question of smoke abatement is a live one in Cincinnati just now.

Fourth.—Mr. Hill's method of determining the accidental moisture in coal by drying a sample on the man-head is a good one. Great care must be taken, however, to insure that a representative average sample is secured. A still better plan is to air-dry, in a warm room, the entire amount of coal to be burned, whenever it is possible to do so.

The above criticisms, it will be noticed, are on details of minor importance, and do not affect the value of the paper as a whole. In the writer's judgment the results are not only strictly comparative, but their absolute value is great, even though the author's methods may not in every detail meet the requirements of the highest technical refinement.

Mr. Hill has done well to call attention to the effect of external conditions upon the quality of the steam. When a boiler is crowded beyond its rated capacity, or when its rate of work is changed suddenly and materially, or where the condition of the fires is altered suddenly, it is natural for the effect to be felt in increased moisture in the steam. It is also true, however, that a well-designed boiler will keep these disturbances at a minimum.

The surprising fact that increased stack temperatures are sometimes accompanied by higher boiler efficiencies has been noticed by others, and is probably due to the fact that combustion is more

Mr. Bryan. complete when the fire-box temperatures are high, and that the heat transmission to the water is better on account of the greater differences of temperature. It is also probable that a much smaller excess of free air is admitted to the grates at such times.

Mr. Carpenter. Mr. R. C. Carpenter (by letter).—In connection with the laboratory work at Sibley College, Cornell University, various forms of coal calorimeters have been used, and the results obtained compared with different calorimeters, and also in a number of cases the determinations have been compared with the computations made from a complete chemical analysis.

As a general conclusion it is probably possible to get accurate results with all the commercial forms which admit of standardization. In fact, it is quite possible to get identical results, by proper management and correct manipulation, with any of these instruments. The question of inaccuracy cannot be raised so much with reference to the instrument used as to the personal habits and methods of manipulation, as it is found that nearly all the errors in calorimetric determinations which are made by students are due to personal errors of observation. The errors most likely to be made are those pertaining to the weighings and to the determination of moisture which may occur in the samples of coal during the manipulations.

The various errors which are made in calorimetric determinations, other than those pertaining to the instrument, may be summed up as follows:

(1) Errors in sampling.
(2) Errors in weighing the sample or the residue of ash, etc.
(3) Errors caused by change of moisture in the sample during the various manipulations.
(4) Errors in weights of water in the calorimeter.
(5) Errors in rise of temperature of water.
(6) Instrumental errors.

The writer will consider these various errors in the above order.

(1) Regarding the errors in sampling, the writer's experience has shown that if the directions for obtaining the sample are properly followed, little or no error should result from this cause. The practical proof of this assertion is the uniformity of results which have been obtained in the laboratory work referred to with a great number of samples of coal, taken not only from the same carload, but from different carloads from the same mine. In fact, these investigations rather serve to show a decided uniformity in the value of the combustible matter of the coal taken from an entire region. The only prominent exception to this is the coal from the Clearfield District, which seems to vary widely in character. The coals from most of the other districts, while differing greatly from each other, agree very closely when from the same mine. Thus there is reason to expect

that a coal from the Lehigh anthracite region will show very closely 14 500 B. T. U. per pound of combustible; the coals from the districts further west and which are classified as semi-anthracites, usually give a heating value of nearly 15 000 B. T. U. per pound of combustible. This value is also very closely realized by the bituminous coals in the Pittsburg district. The coals found in about the same latitude as Pittsburg show a gradual falling off in heating power as the source of supply is removed westward and an increase in amount of moisture. The Hocking Valley coals have a heating value of about 14 500 B. T. U. per pound of combustible, while those still further west show a considerable falling off in value; some of the Illinois coals showing as low as 12 000 B. T. U. per pound of combustible and 8 000 or 10 000 B. T. U. per pound of coal. This regularity in the quality of coal from the same district neutralizes very greatly the effect of errors of sampling. Mr. Carpenter.

(2) The sample usually selected for calorimetric determination has a weight of about 1 gramme, approximately $\frac{1}{250}$ lb. It is not feasible to burn very much larger samples of coal because of the expense of the necessary oxygen, nor is it necessary, provided the weighing be accurately done on a chemical balance. The difficulties of securing perfect combustion would also increase were the sample made much larger. In order to do accurate weighing, a person must understand the use of a chemical balance. This is not a difficult machine to master, but the writer finds from experience that engineers are so accustomed to work with large quantities that more errors are made in its use than in performing any other operation connected with the calorimetric determination, except the temperature readings. There is, of course, no excuse for this condition of affairs, except that engineers are not trained to make fine weighings. Any error in weighing affects the results in a direct proportion.

(3) The samples of coal should be shipped in a hermetically sealed jar, so as to retain the original amount of moisture. Coal gives up or takes up moisture very readily and very rapidly, as it may be drier or damper than the air of the room. This change is so rapid that, if the coal as originally received is burned in the calorimeter, considerable error may result from a change of moisture contents sufficient to affect seriously the results, between the time of taking the first weights and of getting the coal into the calorimeter. This error can be obviated, in large part, by making all determinations on very dry coal. The writer once presented a paper before the American Society of Mechanical Engineers describing the Hygrometric Qualities of Coal.

(4) Errors of weight of water in the calorimeter affect the results directly, since, in most cases, the heating value is determined from the product of rise of temperature and weight of water; the water used is ordinarily from 300 to 800 times that of the fuel burned.

Mr. Carpenter.

(5) Errors in rise of temperature may be due to lack of uniform temperature in the water and to errors in thermometer readings. On account of the large amount of water in relation to the fuel burned, the temperature change is very small, and great care must be taken in temperature determinations. These are fully as likely to occur as errors in weighing. They are minimized by using a very accurate thermometer and taking the readings with great care.

(6) The errors due to the instrument itself and inherent in it can always be eliminated by a process of standardization. The best method of standardizing is, doubtless, the burning of charcoal made from sugar, and this is recommended by the German chemists as the most reliable method of determining instrumental constants.

It appears from the paper that Mr. Hill used for his work the writer's calorimeter; unfortunately, his determinations have been made on coals which the writer has not previously tested, and consequently the writer is unable to give any results which could be used to verify those in the paper. The results seem, however, from a knowledge of coals of adjacent regions, to be reasonable, and what would be expected.

TABLE No. 30.—Approximate Analysis and Calorimetric Determinations, Mechanical Laboratory, Sibley College, Cornell University, November 15th, 1898.

First sample.	Coal used as received.	Dry coal. 100%	Combustible 100%
Moisture	5.68	0	0
Volatile matter	36.17	38.35	43.70
Fixed carbon	46.61	49.42	56.30
Ash	11.54	12.23	0
B. T. U., from calorimeter, per pound	12 016	12 739	14 515
Second sample.			
Moisture	5.78	0	0
Volatile matter	36.29	38.52	43.76
Fixed carbon	46.64	49.50	56.24
Ash	11.29	11.98	0
B. T. U., from calorimeter, per pound	12 047	12 807	14 526
Average.			
Moisture	5.73	0	0
Volatile matter	36.23	38.43	43.73
Fixed carbon	46.625	49.46	56.27
Ash	11.415	12.11	0
B. T. U. per pound	12 031	12 773	14 520

Note.—Coal from Wills Creek, Ohio, used in test of Wickes boiler at Wyandotte, November 15th, 1898. Sample of ash contained 32.9% of combustible, corresponding to 17.3% of ash in the test.

Mr. Carpenter.

Furthermore, judging from the method of use of the instrument, and of the care and accuracy with which the experiments were performed, the results can be adopted as accurate and reliable. In numerous cases the writer has checked the values obtained with the calorimeter used by Mr. Hill with those obtained on the bomb calorimeter, commonly called Mahler's, and when the experiments have been carefully performed, no error exceeding 0.5% was found. This is about the limit of error in the operation of the instrument. As both of the instruments mentioned were standardized and operated by entirely different methods, the concurrence of results would indicate the substantial accuracy of both.

Table No. 30 gives the results of a determination of two samples of coal from Wills Creek, Ohio, made with the Carpenter calorimeter, together with a computation of the heating value from a complete chemical analysis. The results are interesting as showing the close agreement which may be obtained by entirely different methods.

Heat Value of Coal by Computation.—A complete analysis of this coal shows the following constituents of the volatile matter:

H 4.65%; O 9.40%; N 1.44%; S 1.43%.

Combining this analysis of the volatile matter with the approximate analysis we obtain:

Volatile matter, 20.31; moisture, 5.73; ash, 11.415; fixed carbon, 46.625.

The heating value in B. T. U., corresponding to the analysis, may be calculated by the formula

$$h = 14\,650\ [C + 4.28\ (H - \frac{O}{8})] + 3\,740\ S$$

By substitution

$$h = 14\,650\ [66.935\% + 4.28\ (3.48\%)] + 53.6 = 12\,038.7$$

The calorimetric determination for this sample, which was used November 15th, 1898, gave 12 031 B. T. U. per pound of coal.

The close agreement of the computation with the calorimetric determination indicates the correctness of the determination as given. The coals used on two different days from different carloads have heat values which differ from each other by less than 0.5%, and, hence, for practical purposes of comparison, they may be considered identical.

The writer has nothing to urge in relation to the advantages or disadvantages of any particular form of calorimeter, although doubtless much could be said in favor of or against any particular type, depending upon the standpoint from which the subject was viewed. He feels positive that the instrumental errors of the calorimeter used by Mr. Hill are as small in magnitude and as few in number as in those of any instrument made.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE ARTIFICIAL PRESERVATION OF RAILROAD TIES BY THE USE OF ZINC CHLORIDE. Discussion.*

By Messrs. OSCAR LOWINSON, H. S. HAINES, J. I. BOGGS, B. M. WAGNER, GEORGE W. NOYES, JOHN FINDLEY WALLACE, E. E. RUSSELL TRATMAN, C. STANLEY CHURTON, and VAN VRYBERGHE DE CONINGH.

Mr. Lowinson.

OSCAR LOWINSON, Jun. Am. Soc. C. E.—There is no question of greater importance to the engineer using wood, under conditions subject to decay, than that of the preservation of timber. A technical knowledge of the composition of the timber, of the chemical and mechanical conditions to be met, of the effects of moisture and frost, of wear and tear, of the juxtaposition of other materials, such as iron, steel or zinc against wood, of the resistance of the material to transverse crushing, rust or rot, and the relative cost of the operation, etc., is requisite for a satisfactory answer as to whether any method, or which method, is advisable for the preservation of timber.

Owing to the great expense and trouble of making continual physical and chemical analyses, the almost impossibility of examination after application, the long period of time required to determine the success of a method, and the necessity of having perfect confidence in the integrity of those who supply the materials and apply them,

*This discussion (of the paper by Walter W. Curtis, M. Am. Soc. C. E., printed in *Proceedings* for April, 1899), is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to September 23d, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

the use of some method of preservation of timber is, and for years will be, only on works where methods of investigation are carried out with care, or where the user places perfect confidence in the integrity and knowledge of the parties applying the process. Failure is due to ignorance and corrupt practice as much as to fundamental error of theory. Mr. Lowinson.

As the author states, there has been practically no English and little German literature on the subject of value, since the report of the Committee of this Society in 1885. Such knowledge as has been obtained is in the note books of the chemists and engineers who have investigated the question. The various methods of application of preservative processes have had an almost uniform experience. "The history of engineering is strewn with wrecks of numerous methods which promised much," but, under conditions of experience, performed little. Very nearly every large railroad in the United States has at one time or another experimented in this line, and many have ultimately given up, either because of the poor results of the experiment, or for reasons beyond the control of their engineers or chemists. There has been practically no advance in this department of applied science.

"The substances which thus far have given the best results are the application of chloride of zinc, the bichloride of mercury and the products resulting from the destructive distillation of coal tar" (used in creosoting).

The literature of Preservation ably discusses these processes as applied previous to the time of the above-mentioned report. A careful reading of the author's paper will show practically all the defects and objections to the first and second processes. It will only be necessary here to add the practical difficulties which have hitherto prevented the more general application of these processes.

Practical Considerations Affecting the Adoption of Preservative Processes.

1. The Consumer.—In the everyday life of railroad maintenance the "powers that be" are appointed in most cases because of their ability to give their road a good showing on the stock market. The necessity of showing at all times a maximum excess of receipts over expenditures generally influences these representatives of the stockholders to object to the added expense of a plant, or the added cost of preserving the timber, which must thereby be entailed for the period of years until the entire track has been placed in good condition; even on the assumption that the process will be a success. It is assuming that an official would not be true to the trust held if he should increase the expense of maintenance for a single year and thereby depreciate the earning power of the invested capital, even

Mr. Lowinson. though he might be certain of ultimate saving in annual outlay; because of the very condition, *viz.*, the property is a marketable one, and the only practical method of conducting the work is to make only such repairs as are absolutely necessary to prevent its ruin. The results of this process are seen in many large railroads. The only time of actual outlay is at the time of reorganization, or when the road passes into the hands of a receiver to whom the ultimate preservation of the property is of more importance than the question of temporary value of stock. In other words, "it is the duty of the official to run the road on a poverty basis even though at much greater ultimate expense."

2. *The Process.*—Besides those described, the creosote processes are the only others with which satisfactory results have been obtained. Referring to the report of the Committee of this Society, the statement, that creosoting is the most perfect method of preserving timber, still holds true. Since the publication of that report, and as the result of the investigations by the German Governments on their railroads, the attention of scientists has been devoted to the production of creosote and other products in such combinations as to be applicable without the expense of a plant for impregnation under pressure. The present commercial standing of the creosote process itself is also unsatisfactory because of its unsuccessful application in many cases, due either to insufficient impregnation or to the use of an inferior material. It may not be improper in this discussion to state the kind of creosote to be used and its composition.

In the first place, the creosote to be used is what is commercially known as coal-tar creosote. It is one of the many products derived from the destructive distillation of coal-tar. From this are derived products covering almost the entire range of human application. Among these may be mentioned all the aniline colors, the various perfumes, essences and flavoring extracts, high-power explosives, medicines, fertilizers, saccharines, photograph developers, benzines, ammonias, antiseptics, lamp black, lubricating oils, varnish, creosote, shellac, cresylic acid and other wood-preserving chemicals, etc. Besides this material, there is a wood creosote, derived from the destructive distillation of pine, of which more will be said later.

The creosote must be a heavy dead oil, of the best quality, free from tar and other impurities. The ease of adulteration of this product by a mixture with naphtha, and the difficulty of detecting such adulteration without continued analyses of the material, are the probable causes of failure in most attempts where it was assumed that the genuine product was used. So extensively is the adulteration carried on that few engineers have confidence in the process because of their inability to secure this product. It is the cost of this creosote that prevents its practical application. It is necessary to know the

constituents of the coal-tar product, and, at the present time, commercial reasons require that it be brought from Europe. Its composition depends upon the conditions of its manufacture. It must be dense, and have a specific gravity varying from 1.05 to 1.20. It must distil at a temperature of not less than 425°, and must be rich in its principal ingredients, naphthaline (C_{10} H_8) and anthracene (C_{14} H_{10}). Mr. Lowinson.

The method of impregnation, when done by outside contractors, is such as frequently prevents the inspector from knowing whether the proper material is really being injected. It is not the speaker's intention to question the integrity of any plant, but the experience of many engineers has been that deception has been practiced here to such an extent as either to drive out of competition many reputable concerns, or to so discourage the user as to cause him to despair of success. Not only is this true, but the chemical analysis of the material under inspection is a difficult process. These objections are applicable to all processes of impregnation. Properly executed, however, the results are gratifying to the engineer.

The New York Division of the Erie Railroad, 10 years ago, had ties Burnettized, and whereas, untreated, their average life was 8 years, most of these treated ties had to be replaced within 6 years. The zinc-tannin process, with the addition of glue, appeals to the thoughtful engineer as meeting most of the conditions required for a preservative.

3. The Material.—The speaker would ask the author what the depth of penetration is and what method, if any, has been adopted for the protection of the tie when cut or bruised so as to admit water and frost; also the effect of checking and the effect on seasoning of timber.

The speaker does not agree with the author that the transverse crushing is a negligible quality. A series of tests made by the speaker* on the transverse crushing strength of timber gave the following ultimate values: For seasoned white pine, 550; yellow pine, 774; and oak, 1 200 lbs. per square inch.

In an interesting report on the "Strength of Bridge and Trestle Timbers" by a committee of the Association of Railway Superintendents of Bridges and Buildings, of which Walter G. Berg, M. Am. Soc. C. E., was chairman, October, 1895, in regard to transverse strength, the Committee reported and recommended as the average safe compressive strength across the grain:

Southern yellow pine.....	350	lbs. per square inch.
Chestnut and Norway pine.	250	" "
Spruce, cypress and cedar.....	200	" "
Oak	500	" "

The stresses to which a tie is subject, and which in turn contribute to its destruction, either through mechanical force or decay, are the

* *The Engineering Record*, December 12th, 1896.

Mr. Lowinson. transverse crushing resistance, torsion due to track moving and deflection of rails, the shock due to hammering by the live load as transmitted through the rail, and the elements. The results of the speaker's investigations have led him to recommend the adoption of a tie plate between the rail and the tie whenever a preservative process is to be applied in any tie other than oak, for the very reason that although the treatment for preservation will aid in lengthening the life of a tie the decay is only one of the causes of destruction; and protection must be sought in every possible way. When it is seen that the safe transverse crushing strength, for instance, is only 250 lbs. per square inch, even allowing for alleviation due to stiffness of the rail, the speaker would urge as an additional improvement that the tie plate be used.

In regard to cutting treated timber, although not so often met with in railroad ties as on street railroad or dock work, the speaker fears that it is not practicable to treat the timber in the way stated unless there be some arrangement for treating the exposed surfaces.

In the case of creosoted timber, the speaker has seen both the hot and cold dead oil of creosote applied with a brush. A street railroad in New York City is using a 6 × 6-in. "creosoted" yellow pine stringer under its heavy rails, supported on the yokes about 5 ft. apart on centers, and in a large number of cases it is found necessary to notch the stringers at the yokes. The speaker is not aware that the notched portions are undergoing any treatment, but whether because of this the creosoting to which they have been subjected will increase the life of the timber or not time alone can tell.

Liquid Preservatives.—The necessity which has arisen in European countries for treating timber has brought about a considerable amount of experimenting in order to find a substitute for the expensive method of creosoting. There are two reasons for the great cost of treating by creosote. These are:

First.—The high melting point of the naphthaline (175° Fahr.) and the anthracene (210° Fahr.), which are the principal preservative chemicals of creosote.

Second.—The necessity of a pressure process to complete the impregnation.

There are now on the market in Europe at least two materials by the use of which it is claimed that these objections are done away with. Whether the statements of the manufacturers are true or not, the speaker is not yet prepared to state. He certainly would recommend the most exhaustive examination before adopting them. In one case it is claimed that by the addition of zinc chloride to the creosote (a combination of Burnettizing and creosoting), before the completion of the process of distillation, the chloride combines with the naphthaline (C_{10} H_8), to form yellow heavy oils, dichloride of naphthaline (C_{10} H_6 Cl_2) and chloride naphthaline (C_{10} H_7 Cl); and the anthracene

(C_{14} H_{10}) is converted into dichloride-anthracene (C_{14} H_8 Cl_2) and chloride of anthracene (C_{14} H_9 Cl), which at temperatures of 100° to 200° Fahr. possess great viscosity and consequent penetrating power. The writer has seen some remarkable experiments as to the penetrating power of this material. Mr. Lowinson.

Wood Creosote.—Wood creosote is a material obtained from the destructive distillation of pine. Its vital principle is paraffine as distinguished from the naphthaline and anthracene of the coal-tar creosote. A large number of so-called preservatives consist of this material, either by itself or in combination with other materials. It is not looked upon favorably as a preservative, because of its unstable composition and its solubility. The much-advertised creosote and other paints and shingle stains have this material for a basis. It is the speaker's opinion, however, that the addition of a paraffine to a paint adds nothing to its preserving power.

H. S. Haines, M. Am. Soc. C. E.—By far the largest part of the cost of operation of a railroad is devoted to wages and salaries, but if the part devoted to supplies be examined it will be found that the principal expense is absorbed in two items, fuel and timber. When the amount of ingenuity and time which has been devoted to devising means and methods for diminishing the cost of fuel per mile run is considered, and how little ingenuity has been devoted to endeavoring to extend the life of timber, it is evident that the two items are not held in proper proportion. There is hardly any subject that can come before this Society that has more economic value than the preservation of timber; and therefore the speaker has examined this paper with interest to see what could be learned with reference to the general question of the preservation of timber, but confesses to a good deal of disappointment in finding that substantially no advance in knowledge on this subject has been made since the Committee of this Society made a report on the subject in 1885. The conclusion of that Committee was substantially that "creosoting was the most perfect process to use, but in view of the expense it may be preferable to use a cheaper process. With our present knowledge we believe the zinc-chloride process is the advisable one to use for ties in this country." Mr. Haines.

By reading this paper through it will be found that in the year 1899 we are no farther ahead, and know no more than in 1885. In fact, it is a still further disappointment to see that this paper, which has been prepared apparently with a great deal of care, the author having endeavored to gather all the information he could find on the subject, presents so little. It is all impressions, traditions and legends, with scarcely any substantial facts, except in reference to what Mr. Kruttschnitt has done. The speaker knows that Mr. Kruttschnitt is very reliable in all his statements, but unfortunately his statements only cover the period from 1894 to the present time.

Mr. Haines. In a mile of track there are about 2 600 cross-ties, costing, in the track, nearly 50 cents apiece, and their average life is about five years, which represents about $230 per mile per annum. If the life of these ties could be extended, doubled, for instance, the value of any process which would accomplish this can readily be seen.

Reference is made to what has been done in Europe, where certainly a great deal more attention has been paid to the preservation of timber than in America. The speaker's experience has been only with creosote, but extends over many years. Probably no one in this Society has had more extended experience in the use of creosote for timber on railroads than the speaker, and his conclusions are the same as those of the Committee, that it is to-day the most perfect process that can be used. The best evidence of the truth of this statement is that abroad, where the zinc-chloride and also the creosote treatment have been used for many years, they have come back partially to creosote; that is, they are using zinc chloride and creosote in combination, apparently with the view of getting a cheap substitute for the latter.

As to the use of creosote itself, of course every one has had disappointments. The speaker has had some very bitter ones. Mr. Lowinson is right in saying that if the creosote contains certain constituents, particularly naphthaline, and if it is used in certain ways, the result will be just as certain as anything known in our present experience of treating timber. There is not any doubt of it. But the so-called wood creosote is a fraud, nothing else, and any substance that is soluble in water is useless for the preservation of timber, just as anything that is proposed to be superficial in its application is of but little value, for the reason that wherever a notch is cut in a piece of timber a surface is exposed to decay and generally in some vital part of the structure.

This paper relates to zinc chloride, about which the speaker knows nothing; but if the subject of the preservation of timber could be discussed thoroughly and properly and with special reference to creosote as a standard, then conclusions could be reached which would be of great value, not only to railroad companies, but in every other department of industry where timber is being used; and for that reason the speaker wishes that the Society would take up this subject again with one of its committees and bring the state of the art up from 1885 to the present time, as it should be brought, in an authoritative way.

It would be useless to discuss creosote, as the results obtained with that substance are well known from experience in England and in America. The preservation of timber that is properly creosoted is almost indefinite if the proper timber, the proper substance and the proper treatment are used. But this is a question of a cheaper substitute, and the evidence here seems to be that all the cheap substances have been failures, except, perhaps, in Mr. Kruttschnitt's experience

since 1894, and, of course, 5 years is too short a time in which to judge of the results. The most optimistic statement in the paper is that the life of ties is 12 years. If considerable is added to the cost of the material by the zinc-chloride treatment, and if then there is some doubt as to whether the life of the tie will be increased to 9 or 12 years, when the untreated timber would have lasted from 4 to 6, a discussion of the subject has not much practical value. Mr. Haines.

The railroads have not adopted creosoted ties to any great extent, because the users of the timber listen to everybody who comes along. Such persons have all sorts of quack treatments that are ostensibly a good deal cheaper than the real, substantial creosote treatment, which is an expensive one; and the man who is responsible for the annual results does not care to make investments, the benefits of which are to be extended over an indefinite series of years.

The speaker's treatment of timber was not so much with a view to preservation from decay as to preservation from the teredo; and in preserving wood against the teredo, of course a great part of it was exposed to the air and to moisture, in a very hot country where there is much rain. In that way the speaker learned a great deal about the value of the creosote process. In order that the creosoting may be successful the timber and the creosote must be of the right kind and must be subjected to the right kind of treatment.

The creosotes are all hydrocarbons. The lighter parts of so-called creosote are of no value, simply because they are lighter than water, and anything that is lighter than water is soluble therein; but a certain proportion of these lighter parts must be retained, in order to keep the creosote fluid, so that it can be used under pressure. To be of any value creosote cannot be used in any other way than under pressure, and certain conditions must be complied with. The wood must be taken while it is green, and the sap exhausted while the ducts are all open. The creosote must be put in under a suitable pressure until the timber weighs a sufficient amount per foot. Mr. Lowinson has said very properly that a great deal of the value of this process depends on the integrity of the persons who conduct it. There must be some means of inspecting the timber when it is used and of knowing that it is what it is represented to be. With creosote it is entirely practicable to know that what has been paid for has been obtained.

The speaker wishes to call attention to the little progress that has been made in wood preservation, to judge from this paper, since 1885, and, further, to the fact that the subject is a broader one than the mere treatment of cross-tie timber by the chloride of zinc process; and it is to be hoped that this Society will take up the whole subject again and bring up the state of the art from 1885 to the present time. If that can be done the author will have accomplished a great deal by bringing the matter to the attention of the Society.

Mr. Boggs. J. I. Boggs, Assoc. M. Am. Soc. C. E. (by letter).—This paper is both interesting and instructive. In the year 1877 a number of creosoted pole pine ties were put in along the line of the Houston and Texas Central Railroad. During the years 1897 and 1898 the writer had charge of the tracks on the Northern Division of this line, and in the course of his duties had to relay about ten miles of steel, a number of these ties being in the portion so relaid. They were examined closely, and found to be in an excellent state of preservation after twenty years of service, hardly 5% of the total number being rejected as worn out and useless. The most serious objection to them was that the rail cut into the soft wood. This, however, can be entirely avoided by the use of a good, substantial tie plate, and the writer feels no hesitancy in saying that fully 80% of the original number are still in service.

Various kinds of ties were in use along this line during the years intervening between 1890 and 1899, such as white oak, post oak, cedar, Bois D'Arc, creosoted pole pine and Burnettized sawed pine ties. These latter ties were small, being only 6 ins. x 8 ins. x 8 ft., and the great difficulty, as stated by Mr. Mahl in his report, was that they shattered badly under the heavy traffic to which the road was subjected.

No material record of the life of ties was kept before the latter part of 1897, and in the case of the Burnettized ties the personal observation of the writer was such as to lead him to believe that fully half the ties reported as rotten by the section foremen were shattered instead. He has examined the wood in the ties that had splintered, and the original timber was found to be in an excellent state of preservation in a number of cases, although it was broken and worthless. The writer does not believe that the solution of the problem will be found in the weakening of the zinc chloride, but in the adoption of a better class of material than that heretofore used. The same objection applies to these ties as was stated in the case of the pole pine, *viz.*, that the rail cuts into the soft wood very badly, sometimes as much as 2 to 2½ ins. Six years is about the best life that could be obtained on this road, those placed in 1891 being practically all gone in 1898; but, as before stated, the reports of the section foremen on this subject are entirely unreliable, and can in no measure be depended upon as to the causes for removal.

While the first cost of the creosoted pole pine tie is much greater than the Burnettized tie, a few years' service will readily demonstrate that it is by far the more economical.

Mr. Wagner. B. M. Wagner, Assoc. M. Am. Soc. C. E. (by letter).—During 1890 the writer visited the impregnating plant of the Swiss Nord Ost Bahn and jotted down the following notes which may be of interest.

Zinc chloride is used. The plant consists of two cylinders, each

1.9 m. in diameter and 14 m. long; mixing tanks, each holding about 6 000 l., a 15-H.-P. steam plant and various buildings. This plant is said to have cost 120 000 francs. The cross-ties are run in on rail wagons, shaped to conform to the cylinders, each holding about forty cross-ties. Mr. Wagner.

One hundred liters of zinc chloride, containing about 25% of metallic zinc are put in each of the mixing tanks and the tanks then filled up flush with water. The cost of impregnating one cross-tie is stated to be from 0.45 to 0.48 franc.

The plant is never idle as the company is overrun with orders from outsiders. The charge is 10 francs per cubic meter of wood impregnated, while the cost, including interest on capital, sinking fund, etc., is only 4.80 to 5.20 francs per cubic meter.

The quantity of zinc chloride used per cross-tie is as follows:

Oak, 8 kls.; larch, 15 kls.; beech, 20 kls.; pine, 25 kls.

The life of ties is as follows:

	Green.	Impregnated.
Oak	15 years.	20–25 years.
Larch	9–10 "	15–17 "
Fir	7– 8 "	12–14 "
Pine	4– 5 "	9–10 "
Beech	2– 3 "	10 "

Mr. George W. Noyes (by letter).—There are several items which the author has dwelt upon, which are of vital importance to the process. The writer, having had ten years' experience in the operation of the Atchison, Topeka and Santa Fé Railway Company's timber-treating plant, may be able to explain more fully why results in some cases have not been what might have been expected. Mr. Noyes.

While it is the intention of this company to use nothing but fall and winter-cut ties, it is a fact that many thousand summer-cut ties, in which the fungi had already attacked and partly destroyed the fibers, have been treated, as well as those which were practically worthless at the time of treatment. It is the habit of the native tie choppers to go into the timber in the fall and winter, and fell a sufficient number of trees to fill their sub-contracts, making the ties later on. Many of these trees are allowed to lie from one to two years with the bark on. Then they are made into ties, brought in and received as first class and treated, when they were really worthless, as the fungoid life had already destroyed the fibers of the wood. To these two classes of ties can be attributed a large percentage of the tie failures, in the past, and those which will occur for some time to come, as it was only last season that the writer succeeded in stopping this practice.

The proper seasoning of timber, both before and after treatment, is of the utmost importance. To this can be attributed, in a great

Mr. Noyes. measure, the success of the 1885 ties. On account of the solubility of the zinc salt, it is necessary that all the water injected with the chloride be evaporated before the tie is inserted in the track, otherwise the moisture in the soil at once begins its work of leaching out the zinc salt, and the artificial leathery pellicles formed by the glue and tannin are of no benefit, as they have not had time to dry and harden sufficiently to offer any resistance to outside moisture.

The quality of the zinc chloride has much to do with the success of the operation. The company's first purchases of fused chloride, from 1890 to 1895, were supposed to contain from 3 to 5% of impurities, mostly moisture, but these were, in fact, much greater. In 1894 some chloride was found to contain as high as 2.5% of iron, and its effect is already being realized, as a great many ties treated in this year are entirely gone now. From 1885 to 1890 there were no less than eight different persons in charge of the plant. They were not familiar with the process, and by the time they were the works had closed, and they had sought other employment. This, the writer thinks, was detrimental to the process. It must also be remembered that the first treatment was with a 0.015% solution, injecting only about 0.25 lb. zinc chloride per cubic foot, while the more modern practice is to use nearly a 0.03% solution, injecting about 0.50 lb. of chloride per cubic foot, which should make the ties remain antiseptic for a greater length of time. The following is the record of some 1885 ties treated at these works, and laid in the main line just north of the Kaw River Bridge in North Topeka, Kan.: Of 305 ties put in track, 152 were on a tangent and 153 on a 4° curve. In October, 1895, all of those on the curve were taken out, not so much on account of rot as wearing under the rail and respiking, which had made their general condition such that the spikes would not hold. Of those on the tangent, all were in on this date, but 22 were to come out, leaving 130 in the track. On October 31st, 1896, there were still 81 in on the tangent, apparently good for several years. The average life of oak ties at this point is 7 years.

As to the Southern Pacific ties: In the fall of 1896 the writer spent some time in examining ties in their track, and, while the zinc chloride and steaming may be the direct cause of their failure, it is, in his opinion, due to the use of superheated steam (as he understands that they used a superheater during the first few years the works were operated), and to a particular species of timber used, as there are several species of timber that will check along the annual rings or radially if exposed to the sun or dried too rapidly by artificial heat. At the company's works, the only kind of timber treated is red spruce, which corresponds to the Oregon fir, and which treats better when green than dry. This phenomenon is no doubt due to the

resinous nature of the wood, the resin, on becoming dry, forming an impenetrable barrier which the zinc solution cannot pass. Mr. Noyes.

In the summer of 1896 the company treated (with zinc chloride only), for the Burlington and Missouri River Railway, in Nebraska, two sections of ties from the Big Horn Mountains in Wyoming. The original sections were 16 ft. long, half being retained there and sent, to the laboratory at Aurora, Ill. The result of the tests on the treated and untreated timber are shown in Table No. 13.

TABLE No. 13.

Sample No.	Transverse strength.	Deflection.	Crushed under 30 000 lbs.	Spike hold.	Ounces of Zinc Chloride per Cubic Foot.	
					Center.	Edge.
	Pounds.	Inches.	Inches.	Pounds.		
2. Treated	2 766	0.45	0.97	2 017	0.00	12.00
Untreated	2 597	0.48	0.73	1 830	.....	.....
3. Treated	3 125	0.45	0.45	2 600	0.00	27.50
Untreated	3 066	0.54	0.54	2 400	.....	.....
5. New Mexico.. Burnettized.. Pine tie.....	2 975	0.45	0.61	2 775	0.24	6.80
6. New Mexico.. Wheelhouse.. Pine tie.....	2 975	0.38	0.73	2 470	0.62	9.43

Tests of the transverse strength were made on samples 2 x 2 ins. in section, on supports 12 ins. apart. The crushing test was made with samples 6 x 6 ins., by noting the depth to which the flange of a 66-lb. rail penetrated the sample under a load of 30 000 lbs. The figures under "spike hold" show the pull necessary to start a spike which had been driven to a fixed depth. Fine borings from the Wyoming ties were washed in running water for 48 hours, then washed in distilled water and analyzed for zinc chloride. Though they contained 20.8 oz. of zinc chloride per cubic foot before washing they were found to contain none after this treatment.

In the summer of 1896, in order to determine whether the Burnettized or Wellhouse ties retained the more zinc chloride, five ties, which had been treated by the two processes, and which had been in the ground for 6 years, were taken up, and 6-in. blocks cut from each end and from the middle of each tie, making thirty blocks in all. Analyses of ash, as made by Robert W. Hunt & Co., Chicago, showed the average of ten end blocks of the Burnettized ties to contain 0.522% of zinc chloride.

Mr. Noyes.

The average of ten end blocks by the Wellhouse process	0.674	per cent.
The average of five middle blocks by the Burnettizing process	0.496	"
The average of five middle blocks by the Wellhouse process	0.525	"
The average of the entire fifteen Burnettized blocks	0.513	"
The average of the entire fifteen Wellhouse blocks	0.624	"

The general conditions as to drainage, soil, traffic, etc., were the same in both cases.

While this test showed more zinc chloride retained in the ties treated by the Wellhouse process, yet there is one point which has never been touched upon, *i. e.*, the organic decomposition of the glue when not fixed by the tannic acid. This is a very important point, and should be a strong argument in favor of the three solutions as now used by the Chicago Tie Preserving Company.

The fact that it takes a great deal longer for the Wellhouse ties to dry after treatment, proves that the treatment does, to a certain extent, prevent the passage of moisture into the wood.

Mr. Wallace. JOHN FINDLEY WALLACE, M. Am. Soc. C. E. (by letter).—This paper discusses a question which is becoming yearly more and more important to the railroad companies of the United States. The annual increase in the amount of ties used for renewals and the steady diminution of the timber supply are gradually attracting the attention of railroad owners and managers to the tie question.

There seems to be little doubt that, from now on, more and more attention will have to be given to preservation methods, but this is only a step toward the solution of the problem, which must undoubtedly come in the use of some substitute for the wooden tie, and which, it would seem, must eventually compel the use of a metallic tie.

An investigation which the writer has just completed shows that the average life of the untreated white oak tie in northern Iowa is $11\frac{1}{2}$ years, in Illinois 10 years, and in Kentucky and Tennessee and the high ground in Mississippi $7\frac{1}{2}$ years. These figures apply to winter-cut white oak ties grown on the high lands of Kentucky, Tennessee and Mississippi.

Black and red cypress ties, used in the low lands of Mississippi and Louisiana, have an average life of 10 years, and this life can be increased several years by the use of tie plates, as these ties wear out before they decay.

These figures are taken from the actual records of ties used during the 10 years ending 1898, and the writer considers them reliable for the general territory of which the several districts mentioned form a part.

One difficulty met by railroad managers, in considering tie-preser-

vation methods, is the want of accurate and reliable information as to the actual increase in life due to the various processes of treatment applied to the different kinds of material. Generally, the close-grained woods, similar to white oak, are not susceptible of economical treatment. Generally speaking, the woods which absorb a sufficient amount of the preserving material are coarse of grain, and it is usually found necessary to use tie plates with them, at an additional expense of 10 to 15 cents per tie, in order to get the full benefit of the preserving process and prevent the tie from wearing out before it decays. Mr. Wallace.

When new processes are brought before railroad managers, actual facts are necessary for the passing of proper judgment on what is the best and most economical thing to do, under the circumstances. It is very difficult to prevail on railroad managers to enter into any line of expenditure which is more or less experimental. Many railroad managers have given more or less general attention to the question of treated ties, but have, so far, not felt justified in taking practical steps toward the general use of any of the existing processes, on account of the expense and uncertainty as to results, although there is little question but that the necessities of the near future will force action in this direction.

E. E. Russell Tratman, Assoc. M. Am. Soc. C. E. (by letter).— Mr. Tratman.
One of the most important points of this paper is the very forcible way in which it shows that no reliable records exist or are kept in the United States as to the life of railway ties, from which averages of life and cost of different kinds of timber can be computed. Without such records, the calculations and estimates of comparative durability and economy of different ties and different preservative processes, are really little more than guess-work (except for individual roads), as there is no real foundation upon which to base any comparisons. The few railways which keep useful records have generally such diverse forms and methods, that it is difficult to make comparisons between them.

The ties are a very important consideration, in their aggregate cost, the labor of renewal, the disadvantages of frequent renewals (from traffic and economic points of view), and the consumption of forest timber which these ties represent. In view of all this, it is most desirable, even at this late day, to endeavor to have some uniform system adopted by which records may be kept which shall be complete, comparable, and free from the influence of the personal equation which makes present statistics so unreliable as a basis for any general conclusions. It is, of course, difficult to get a system of this kind introduced and properly followed out, but the only way is to make a beginning of some kind, and then gradually develop it on lines carefully thought out.

It seems to the writer, that the keeping of these records upon some

Mr. Tratman. uniform and carefully formulated system, is a duty that might be properly imposed upon the division roadmasters, or officers of similar position. The section foremen should be required to keep very careful record of the number and cause of tie renewals (decay or rail-cutting). The roadmasters should then, from this information and their own knowledge and observation, prepare the broader records. Some means of supervision should also be devised, to ensure that the inspection and renewal of ties and the keeping of the records are done carefully and not in a mere hap-hazard or perfunctory manner.

The following list shows some of the items which should be included, and Table No. 14 is a sample of the suggested form of record. With such complete and uniform records from a number of railways for a term of years, and with the records of the operations of preservative processes and their cost, some broader and more reliable information could be compiled than is possible with existing statistics. This information would undoubtedly lead to greater care in the use of ties.

1. Miles of single main track on the division.
2. Approximate tonnage carried over the division during the year.
3. Kinds of timber used and the percentage of each.
4. Number of ties per mile (according to the standard spacing or standard number per rail length).
5. Preservative treatment of the ties.
6. Number of ties of each kind renewed.
7. Cost of ties of each kind renewed.
8. Date when ties removed were laid.
9. Place where ties of each kind were obtained.
10. Tie-renewals per mile of track.
11. Kind of ballast in which ties were laid.
12. Weight of rail.
13. Are tie-plates used?
14. General condition of track; including condition of rails, and road bed, surface, line, etc.

TABLE No. 14.—Tie Record, Smithville Division, A. B. & C. Ry.

1.....	50 miles.		
2.....	.....tons.		
3.....	Oak, 50%.	Pine, 25%.	Hemlock, 25%.
4.....	2 464.	2 816.	2 816.
5.....	None.	10% zinc chloride. 90% none.	10% creosote. 10% zinc chloride. 80% none.
6.....	5 000.	3 125.	3 750.
7.....	55 cts.	50 cts.	20 cts.
8.....	2 000 in 18..... 3 000 in 18	2 000 in 18..... 1 125 in 18.....	3 000 in 18..... 1 750 in 18.....
9.....	Pennsylvania.	Georgia.	Pennsylvania.
10.....	200.	250.	300.
11.....	Stone.	Screened gravel.	Common gravel.
12.....	80 lbs.	60 and 70 lbs.	60 and 70 lbs.
13.....	No.	At joints.	On all ties.
14.....	Good.	Good.	Rails worn, and surface poor.

Mr. C. Stanley Churton (by letter).—On reading Mr. Curtis' able and exhaustive paper one is struck by the extremely short life of the ties. This, however, is partly explained when he states that "they perish by the mechanical destruction of the fibers under the rail," and also by the cutting out of the fibers in the process of respiking. Further on it is shown that ties can be protected by plugging the spike holes, and the writer would suggest that before insertion the plugs be dipped, either in liquid creosote, or a solution of chloride of zinc, or sulphate of copper. Mr. Churton.

The whole question of preserving timber by chloride of zinc seems hopelessly mixed. For instance, Mr. Curtis, in remarking on progress since 1885, writes: "Three plants treating timber with zinc chloride were in operation in the United States, each working with a different process," and further alludes to "the method adopted to prevent the impairment of the preservative treatment by the washing out of the zinc salt, which is quite soluble in water."

The paper does not state how one process differed from the others, but the writer thinks that Mr. Curtis is wrong in his statement concerning the washing out of zinc salt. The fact is—and it is an important fact, and one not to be forgotten—that chloride of zinc acts on vegetable fiber the same as iron acts on cotton or linen, forming what is known as iron-mould. The cotton can take up only a certain quantity of iron, but this becomes incorporated with it, and no amount of washing or boiling can take it out; there is a chemical combination between the iron and the linen. An excess of iron would rot and destroy the linen, and the excess which could not be combined with the linen could be washed away.

The fiber of the wood can enter into combination with the zinc up to a certain amount, but more than that would be worse than useless, in fact, would actually defeat its own object and hinder rather than assist the longevity of the timber.

Again, chloride of zinc attracts water from the atmosphere, therefore an excess of chloride would keep the ties perpetually wet; in fact they would never become perfectly dry; consequently decomposition of the fiber under the rail would be hastened. It is admitted on page 237, that on the Southern Pacific Railroad, the quantity of chloride used was too great.

In some parts of Mr. Curtis' paper, the process is described as being performed under pressure, at others in a "vat," when the solution was raised to a great heat, 250° to 300°. The writer's opinion is that the life of the wood is in the sap; destroy the sap and the life is destroyed. The best way to preserve timber for railway use is to let it be well seasoned before being placed under pressure. The actual drying, under favorable circumstances, does not take very long

Mr. Churton. after 10 lbs. of solution of zinc chloride of 1.025 specific gravity has been forced into it; when dry it is harder, denser and more durable. The use of the word "vat" gives one the impression that in some places ties have been preserved by being macerated in a hot solution of zinc chloride; and that great heat has been used. The timber has, in fact, been boiled in a preservative solution. The object of applying heat—which is not absolutely necessary when timber is treated under pressure—is to expand the pores and to hasten the absorption of the preservative, and the application should be continued only as long as is necessary to absorb the quantity of solution required. To destroy the sap by excessive heat, in fact, by boiling it out, is, in the writer's opinion, a positive mistake.

It cannot be too strongly insisted upon that many theorists, faddists and experimentalists have, from time to time, invented processes for the preservation of timber, which have only had the effect of destroying its vitality.

About twenty-five years ago a process unknown to the writer was much advocated, but after a little time the wood, some specimens of which were shown ten years ago, broke like carrots. Timber, like wine, requires careful and skilful handling, and those whose duty it is to treat it with chemicals should understand thoroughly the nature of the chemicals used.

The late Sir William Burnett, the inventor of Burnettizing, was a medical director in the navy, and was cognizant of the therapeutical effects of chloride of zinc on vegetable fiber. His directions were that one part of liquid chloride of zinc having a specific gravity of 1.600 should be mixed with 40 parts of water, and experience has proved that this is the best strength for treating timber, and most conducive to its longevity.

Mr. de Coningh. Mr. Van Vryberghe de Coningh* (by letter).—On the Dutch State Railroad wooden ties have been used exclusively since 1889, as steel ties did not give satisfaction. Baltic pine has been used on tangents and curves of greater radius than 1 000 ms., and Dutch oak or beech for curves of less radius and for switches. Dutch oak is preferred to Baltic oak because having grown more slowly it is consequently tougher, though less showy. Up to the present time no difference has been observed between inland and foreign beech. The ties are from 2.60 to 2.70 ms. in length. Their cross-section is as shown in Fig. 1.†

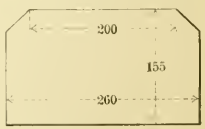

Fig 1.

The rails are of the T type, weighing from 33.7 to 40 kgr. per meter, and are not fixed directly on the tie, but upon steel plates

* *Ingenieur der Staatsspoorwegen*, Utrecht, Holland.

† The dimensions on Figs. 1 and 2 are in millimeters.

(shown in Fig. 2), and these, at the same time, give the desired inclination to the track. The length of these plates is 200 mm. The rail and tie-plate are fixed by spikes. Mr. de Coningh.

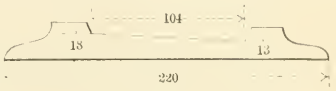

FIG. 2.

Formerly the ties were planed only at the places to be occupied by the plates, by making incisions with a saw and planing between, but these incisions caused the infiltration of rain water, and at present the whole upper surface of the tie is planed.

No preservative process is applied to oak ties. Pine ties are preserved by the modified Rütgers process, and beech ties with tar-oil (creosote). In the future, when the installations have been modified, the Rütgers process will also be applied to the oak ties.

All timber is cut in the winter, and great care is taken to have the ties thoroughly air-dried before they are treated, because, in that condition, the wood absorbs the preserving liquids more readily.

As navigation is closed by the ice during winter and spring, the Baltic pine is shipped to the Netherlands between June and December, and is, therefore, generally piled up in the Netherlands during one winter. Only in exceptional cases, and when there is great need for it, lumber shipped in early summer is put through the process in the fall.

Beech shipped in the spring is used in the summer. It is easily impregnated and is then very durable, but if it is not impregnated it decays very soon. Therefore, great care must be taken with it before it is impregnated, for, if it is piled up carelessly, it decays readily.

The modification of the ordinary Rütgers process concerns only the composition of the impregnating liquid, which is 5° Beaumé (4% $Zn Cl^2$) instead of 3° Beaumé (2% $Zn Cl^2$) as ordinarily used. To 30 kgrs. of the mixture (the dose for one pine tie) is added 5 kgrs. of tar-oil, instead of 2 kgrs.

Up to the present time experience has not shown that by augmenting the quantity of zinc chloride the fibers of the wood are attacked, or that the wood becomes brittle. Great care is taken, however, to keep the zinc chloride absolutely free of acid.

The quality of the creosote is fixed by prescription and regularly controlled. The process is wholly that of Burnett, applied by Rütgers: steaming, air-vacuum and impregnating under pressure, the time for each part of the process depending on the degree of moisture in the wood.

Until the impregnated ties are to be used in the track, great care is taken to keep them piled up close together; just contrary to the method in which they are piled before impregnating, which is very loose.

There are no statistics yet as to the life of impregnated ties.

Mr. de Coningh.

The cost, during the last 6 years was as follows:

Pine ties	1.30 to 2 guilders (52 to 80 cents).
Beech "	± 2.70 guilders ($1.08).
Oak "	28 to 38 guilders per cubic meter.

In 1898 the cost per tie was as follows:

Pine	0.46 guilder (18½ cents).
Beech	0.72 " (29 ").

In these prices are included, respectively, 0.30 and 0.58 guilder (12 and 23 cents) for the liquids (the prices of these varying yearly), and 0.07 guilder (2¾ cents) for labor. The remainder makes up the expenses for engineers, coal, repairs to machinery, and supervision.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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PAPERS AND DISCUSSIONS.

Note.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

ON THE THEORY OF CONCRETE.

Discussion.*

By Messrs. C. E. Fowler, G. W. Tillson, H. F. Dunham, E. Sherman Gould, P. W. Henry, Herman Conrow, Allen Hazen, B. M. Harrod, William M. Hall, Ira O. Baker, L. J. Le Conte and H. Von Schon.

Mr. Fowler

C. E. Fowler, M. Am. Soc. C. E.—This subject has impressed the speaker as being a very important one, for the reason that in figuring on work he has, as a rule, found that engineers and contractors know only approximately the quantities of cement, sand and broken stone to order for a given number of yards of concrete of certain specified proportions. The most convenient way of determining these quantities is to prepare tables from which the amounts required for a yard of concrete of any proportion may be taken, using only such proportions as have the voids in the stone entirely or more than filled.

Many writers on this subject have assumed that every concrete, no matter what the proportions may be, has simply a yard of broken stone, without regard to the fact that there is more or less mortar in different proportions. In making up a table of this sort the speaker used the proportions of mortar given in Baker's "Masonry Construction," but found that this did not agree with actual practice. These tables in "Masonry Construction" were made up from theoretical

*This discussion (of the paper by George W Rafter, M. Am. Soc. C. E., printed in *Proceedings* for April, 1899) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv , p. 71.)

Communications on this subject received prior to September 23d, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

Mr. Fowler. considerations, and it is understood that they are to be revised in a new edition.

TABLE No. 4.—Portland Cement Concrete.

No.	Proportions.	Barrels cement.	Yards sand.	Stone, 0.4 voids.	Stone, 0.5 voids.
1	1-2-3	1.77	0.51	0.87	1.05
2	1-2-3½	1.68	0.49	0.91	1.10
3	1-2-4	1.59	0.47	0.95	1.15
4	1-2-4½	1.48	0.44	1.00	1.20
5	1-2-5	1.39	0.42	1.04	1.26
6	1-2-5½	1.30	0.40	1.08	1.30
7	1-3-4	1.30	0.57	0.83	1.00
8	1-3-4½	1.22	0.54	0.89	1.06
9	1-3-5	1.16	0.52	0.92	1.11
10	1-3-5½	1.09	0.50	0.97	1.16
11	1-3-6	1.04	0.48	1.00	1.20
12	1-4-6	1.00	0.55	0.91	1.09
13	1-4-6½	0.96	0.53	0.94	1.13
14	1-4-7	0.92	0.51	0.97	1.17
15	1-4-7½	0.88	0.49	1.00	1.21
16	1-4-8	0.83	0.47	1.03	1.25

About 34 cu. ft. loose = 1 yd. rammed.
Labor = ½ to 2 yds. per man per day.

Table No. 4 was made up some years ago by comparison with actual cases, and has been found to give very satisfactory results in practice. The column for broken stone with 0.4 voids will represent closely ordinary broken limestone which chips up more than a harder stone and consequently has less voids; while the column with 0.5 voids represents trap rock, the break in which is more angular. It will be readily seen from the table for Portland cement concrete that certain proportions will not have the voids filled and should not be used. For example, with 1 to 3 mortar and with stone having 0.4 voids all the concrete given will have the voids filled, or more than filled, while with 0.5 voids only the 1-3-4 (No. 7), will have the voids filled. The table is made up on the basis of shrinkage under ramming of about 7 cu. ft., and 3.8 cu. ft. of cement to a barrel.

In a recent paper presented to the Society, one of the members stated that concrete was not a fit material for piers of railway bridges, unless a pedestal block of stone was used on which to rest the masonry plates. The speaker would refer that member to the engineers of half a dozen large western roads who use concrete for piers and allow the bed plates to rest directly upon the concrete. Concrete is certainly better than most stone, except granite, and probably 50% better than much of the stone commonly used.

Mr. Tillson. G. W. Tillson, M. Am. Soc. C. E.—The speaker has had some experience in the last few years in mixing concrete for street work, by

hand and by machinery. For such work, the layer of concrete is comparatively thin, only 6 ins. thick, as a rule, and the mixer, if a machine is used, must be moved forward to keep out of the way of the finished work. In Brooklyn, for the last two or three years, almost all the concrete for street pavements has been mixed by machinery such as was described by Mr. Gould. The method of procedure, however, is a little different. The sand and cement are first thoroughly mixed dry, in the proper proportions, on one side of the machine. The stone is all delivered on the opposite side and is dumped on a plank platform. Before the stone is used, it is thoroughly wet. A gas pipe, perforated throughout its length and running lengthwise inside the cylinder of the machine, admits the water. Then, when the cement and sand are so mixed as to produce a good mortar, an inspection of the material as it comes from the machine indicates whether or not the stone is in the proper proportion to give a good concrete. Good concrete cannot be obtained without good mortar. The specifications call for a 1, 2, 4 mixture, and, generally, there are two men shoveling in the material on the cement side and four men on the stone side, and the shoveling is continuous, on both sides, until the machine has to be moved. The mixer has a gate at the lower end, attended by a man who dumps the concrete into the wheel-barrows. In that way, with proper care, the result is as good as if the work was done by hand, if not better. In both cases the quality depends entirely upon the work being done properly and systematically. Good concrete can be obtained when mixed either by hand or by machinery. When mixed by hand, the difficulty is, as has been said, to have the mixing done thoroughly. It is hard work, and in warm weather the laborers will get through with a batch as soon as possible. Mr. Tillson.

In making machine-mixed concrete the difficulty is to supply the ingredients in the proper proportions, so that there shall not be an excess of any single material. The stone and cement should be put in at regular intervals. Their supply will be graded very nicely after the men have worked at it a little while, especially if the foreman understands the work. There is great danger, however, in using a machine, if both the foreman and the men are inexperienced.

The relative costs of the two methods, as nearly as the speaker has been able to estimate roughly, by counting the number of men on gangs working by each, are about 7 cents for the machine-mixed and 10 cents for the hand-mixed concrete per square yard, 6 ins. deep. This will doubtless vary 10%, either way, according to the contractor or the foreman.

The speaker has found the amount of water that should be used to be dependent upon conditions. A dry concrete will give good results in cool weather, and the same mixture, in warm weather, will be of practically no value. When the weather is very hot, and when con-

Mr. Tillson. crete has to be exposed to the air, as when used as foundation for an asphalt pavement, unless it is mixed quite wet the water dries out so rapidly, for the first inch at least, that there is no chance for the chemical action to set in. The result is, that the concrete is dry, and, in many cases, is readily disintegrated near the surface, when under other conditions, good results would have been obtained. The speaker believes that it is safer to make a concrete a little wet rather than a little dry, and he had the idea that hand-made concrete is better, in the abstract, than machine-made concrete, but judging from some hand-made work seen recently, he believes it fair to say that the machine-made concrete is just as good as the hand-made, and that its quality, in both cases, depends greatly on the care taken in mixing.

Mr. Dunham. H. F. Dunham, M. Am. Soc. C. E.—The presence of dust in the concrete has been mentioned. In certain parts of the country large quantities of limestone are crushed for use in blast furnaces, and in some establishments the limestone is also crushed for concrete. They have, altogether, a heterogeneous product; some of it is very coarse and is used in mills, some is of fine grades and is used for walks when it can be disposed of for that purpose, some is crushed to pass through a $2\frac{1}{2}$ or 2-in. ring, as the case may be, for regular concrete, and there is also a product, called dust-screenings, which resembles a mixture of half flour and half Indian meal, with some pieces as large as a kernel of corn.

The speaker was desirous to see the effect of treating that dust in connection with concrete just as one would treat sand. He, therefore, made a mixture in proportions of 1 part cement, $2\frac{1}{2}$ parts dust and $3\frac{1}{2}$ parts broken stone of about the size which would pass through a 2-in. ring. A block of about 9 ins. in each dimension was made, kept for a period of two or three weeks and tested. It was loaded to the limit of 3 tons to the square foot, which was more than it was required to bear in the place where it was to be used, and showed no sign of fracture or disintegration. It seemed to be entirely satisfactory. American cement of ordinary grade was used, and to appreciate the condition fully it would be necessary to see the so-called dust. It was material that could not well be shoveled if there was any wind.

In an experiment regarding the voids in crushed limestone, an ordinary metal receptacle was first filled with the large broken stone, 2-in. ballast, and then with water, and it was found that the quantity of water required was substantially the same as that required to fill to overflowing the same vessel after it had been completely filled with this so-called dust; showing that the limestone, whatever its condition —probably it would hold true for any grade between those two— would require substantially the same amount of water to completely saturate it and fill the voids.

E. Sherman Gould, M. Am. Soc. C. E.—In regard to the comparative merits and quality of hand-made and machine-made concrete, the speaker thinks the impression is somewhat general that the hand-made concrete is the better, but he does not know how that has been ascertained, whether by actual examination or by thinking that it ought to be the best because it is hand-made. Hand-made work of all kinds is often looked upon as being superior. The speaker has been led to take a different view, and in his own experience, as far as he has been able to judge, without applying any critical test, such as testing the tensile or crushing strength of large blocks, but just from the appearance of the concrete, he has been led to think that the machine-made product was the better, and it is natural that it should be, because the mixing of the ingredients of concrete, being one of the hardest kinds of work that a man can do, is often slighted. The material must be turned over thoroughly; the shovels must go down to the bottom of the heap until they scrape against the boards of the platform, and the mixture must be picked up from the bottom and turned over again and again. On the other hand, when the product is put in a machine and stirred around mechanically there must be a more complete and regular mixing than could otherwise be obtained. No doubt there are several ways of making machine-made concrete. The only machine the speaker has used is one with a metallic shaft running through the box, with fins or fans working in the material; the box being on a slant and the material fed in at one end and out at the other. The comparisons of hand-made and machine-made concrete were made on work which the speaker was executing, and when, in order to gain time, he supplemented the machine by gangs of men making the concrete by hand in the ordinary way, and using the same proportions as in the machine; the relative merits of the two products were judged in that way. The method of making the machine-made concrete was as follows: The sand and cement were measured out on the platform surrounding the machine and mixed dry, only one batch being mixed at a time. The proportions were 1 of cement, 3 of sand and 6 of broken stone. The stone was crushed in a machine. At first it was screened, but was afterward used just as it came from the crusher, and it was believed that the results were better, or at least as good. A barrel of cement was mixed dry with 3 barrels of sand, and each barrel of stone was thoroughly wet. The stone was emptied out on the platform, and then the stone and mortar were shoveled into the mixer. After a short time the men got accustomed to the rate at which the stone and mortar should go in, so that by the time all the stone was gone, all the mortar was also used up and the bed was cleaned off. The water was fed in and was regulated by a valve managed by a man who stood at the lower end of the machine so that he could see the concrete as it fed out into the iron wagons, by which it

Mr. Gould.

Mr. Gould. was conveyed to the work. After a little while the man who was managing the water valve knew the proper consistency; if the concrete was too wet or too dry, word was sent to him and he regulated the water accordingly. The speaker feels quite sure that no hand-made concrete, using the same proportions, could equal that machine-made product, because of the thorough way in which it was churned up.

Another point, with which the speaker agrees fully, is that although in a laboratory experiment the dry mortar or concrete is stronger, yet in practice it is very apt to be weaker, and although too much water is bad, too little is a great deal worse. That has always been an axiom with the speaker, and he has found it was difficult to get the necessary amount of tamping done to bring up the moist surface on the dry concrete, without which it is worthless and ought to be removed. If the concrete is so dry that tamping fails to bring the moisture to the surface—indeed it ought to be almost a water surface rather than mere dampness—the concrete is worthless and should be taken out. By putting in a little more water, however, this is avoided, the work is certainly done more quickly and, on the whole, a better result is obtained. The speaker was brought up in the dry school, and got his first education in concrete making from General Gillmore on work which he was executing for him in the rebuilding of Forts Sumter and Moultrie, in Charleston Harbor. All the work there was done according to General Gillmore's directions, under rules which he had given. It is certain that a very fine concrete was produced, but it was with great labor, and was done by day's work. If anything was at all wrong the concrete was simply taken out and the work started over again, nothing which was at all imperfect being allowed to remain. The speaker, however, gradually came around to a freer use of water, and thinks that it gives, upon the whole, a better result. One reason is that in making successive beds of concrete, if they are being brought up in 4 or 6-in. layers, which is apt to be the case, the last bed tamped makes a better bond with the next bed put on if it is a little more moist. The speaker is quite convinced that a section through a large mass of concrete which has been constructed of very dry 6-in. layers would be very apt to show partings between the successive beds. That may or may not be, but the speaker would rather expect to find it, and, more than that, he would be afraid that the dry bed had immediately taken a set and that the placing of the other bed upon it and the tamping—the influence of tamping, of course, is felt through the entire bed—would have a rather disastrous effect upon the partially set concrete in the bed below; whereas, if it is more moist the set does not take place so rapidly, and there is not the same danger in pounding on a bed of concrete which has just commenced to take a set. In other words, if every possible precaution be taken and the

work is done under able and constant inspection, it is probable that a little stronger and better product may be obtained with a very dry mixture; yet, taking contract work as it is done all over the country, there is a better chance to get a good result if everything is kept a little moist. Mr. Gould.

After concrete has been placed, a very important detail is to keep it constantly wet until it is covered over with whatever material is to come on it. Concrete is largely used for foundations upon which stone work is built, but in the speaker's practice all exposed concrete is kept wet for an indefinite period. He has had concrete watered every day and kept constantly wet for probably two or three weeks after it had set. In using hydraulic cement, which is the only kind ever used now, the great point is to let it always have plenty of water, and, after the concrete is all finished, no matter how much water has been used in making and placing it, it should be kept constantly wet, not by dashing water on it, but by sprinkling. The longer this can be done, the better.

The experiment mentioned by Mr. Conrow was conclusive as regards that particular hand-made and machine-made concrete, but the speaker does not think that the two can be spoken of so generally as that. Good hand-made concrete would be better than bad machine-made concrete and *vice versa*. The only way to settle the matter satisfactorily would be to test the very best of each kind.

The greater adhesion in the washed bricks was probably due to the saturation with water. If two bricks are washed and thoroughly saturated with water their adhesion is very much greater than that of two dry, unwashed bricks. The speaker would attribute the increased adhesion to the moisture.

P. W. Henry, Assoc. M. Am. Soc. C. E.—About a month ago the speaker saw in New Orleans the work which is being done for the Drainage Commission, of which Major Harrod is Chief Engineer. The machine used in mixing the concrete is the ordinary asphalt mixer. It consists of a chamber in which two shafts provided with teeth revolve in opposite directions and draw the mixture toward the center. The sand, cement and shells are placed in receptacles above, which hold the required amount. The dry materials are dumped directly into the mixer, and are mixed dry until all are practically of one color. The water is then turned on. The speaker thinks that the quantity of water is judged by the eye and not measured, but it could be arranged to admit water by volume. The result is an absolutely perfect mixture, if such a thing is possible; because a great number of analyses of asphalt mixtures with that type of machine have shown an identical grading of the sand throughout all parts of the mixture, and also the same amount of bitumen in the mixture. The results in New Orleans are certainly very satisfactory. Whether this style of mixer would be as suitable for Mr. Henry.

Mr. Henry. trap rock, for instance, which is very heavy, or even for limestone, is a question upon which the speaker is not able to express an opinion. It certainly works very satisfactorily with the shells.

The speaker's experience with dry and wet concrete has been confined entirely to laying concrete for asphalt pavements, and indicates that it is safer to use a moderately wet mixture for that purpose. When concrete is laid during hot weather, it is a very good practice to sprinkle the surface every night until the concrete is thoroughly set, or for, say, eight or ten days.

Mr. Conrow. HERMAN CONROW, Jun. Am. Soc. C. E.—In ten of the tests given by the author for "dry," "plastic" and "excess" concrete, under the same conditions, the speaker finds that the average ratios of strength are as follows:

Dry, 29.1; plastic, 26.6; excess, 25.3.

About three years ago the speaker made some experiments upon concrete with especial reference to its economical mixing and laying. The experiments were made under average conditions of mixing by hand, and the figures here given were obtained with a gang of ten men and a foreman. When laying wet concrete, the gang was divided as follows: One foreman, 9 mixers, 1 rammer. An average day's work was 15 cu. yds. of concrete in place. When laying dry concrete, the gang was divided thus: One foreman, 6 mixers, 4 rammers; and, under the same conditions as above, an average day's work was 8 cu. yds. in place. Taking the foreman's wages at $2 per day, and the men at $1.50 each, the cost of laying 1 cu. yd. under the two conditions was as follows:

Cost of Laying Wet Concrete per Cubic Yard.		Cost of Laying Dry Concrete per Cubic Yard.	
Cement (1 barrel Portland)	$1.50	Cement (1 barrel Portland)	$1.50
Sand	.50	Sand	.50
Stone	1.00	Stone	1.00
Labor	1.13	Labor	2.12
Total	$4.13	Total	$5.12

The dry concrete thus cost 24% more than the wet. From the averages of the previously mentioned ten tests, given by the author, dry concrete is about 15% stronger than wet. Consequently an engineer would not be justified in using dry concrete except in cases where unusual strength was desired, as in arches.

Only those who have handled concrete under actual conditions can realize the greater ease of mixing and laying it when water is freely used. Especially is this true in hot weather. Dry concrete may be an uncertain material, and if the inspector relaxes his vigilance for a few minutes a weak spot is liable to occur in the masonry. Although dry

concrete, by a great amount of ramming, may be made stronger than wet concrete, yet if it is not thoroughly rammed it may possess almost no strength, even when the cement is first class. Wet concrete cannot be weak—provided the cement is good and it is reasonably well mixed. Mr. Conrow.

The speaker also experimented with different methods of mixing concrete. After trying various ways the following proved to be the best. Mix the mortar dry; then wet it and mix until all is of the same consistency and about as wet as can be handled easily with shovels. After the stones have been spread evenly to a depth of about 8 ins. and washed clean, spread the mortar over them and then begin the turning. By using wet mortar it saves at least one turning as the mortar percolates through the stones and three turnings with wet mortar will incorporate more thoroughly than four turnings with dry mortar.

As to the comparative merits of hand-mixed and machine-mixed concrete, the speaker prefers the former, because in most machines there is given no opportunity of thoroughly washing the stone before it is mixed with mortar. The dust rising continually in the machine settles on the stone prior to contact with the mortar, all to the detriment of the concrete. Only by mixing in the open air with plenty of room can the stone be kept perfectly clean until the mortar is applied. The importance of thoroughly mixing the concrete is only equalled by that of having the stones clean.

There is no material used in engineering which is so variable as concrete, and the remedy for a vast amount of poor concrete is the same as in a majority of cases of poor brickwork and weak masonry—the use of plenty of water during construction.

In tearing out some concrete, the speaker noticed that when the hand-mixed concrete was broken with a sledge hammer, it would almost always break across the stone, thus showing that the concrete was about as good as it was possible to make it. With the machine-mixed concrete, however, it often happened that the stone did not break, but broke away from the cement and left a cavity, showing, evidently, that there was weakness at that point. It is well known that if the stone is covered with a film of dust, the concrete will not be so strong as if it is perfectly clean. The speaker does not see how it would be possible, in the machine described by Mr. Henry, where all the dry materials are first mixed together and then wet, to get a perfect concrete. If, for instance, two bricks, as they come from the brickyard, are laid with a layer of mortar between them, and two other bricks, first washed perfectly clean, are similarly laid, the difference in tensile strength between the two would be very great. The same principle applies to concrete. If all the ingredients are mixed together dry in a machine, the case is exactly the same; dust-covered stone is used and the resulting concrete is not so strong as concrete made from thoroughly washed stones.

Mr. Hazen. ALLEN HAZEN, Assoc. M. Am. Soc. C. E.—The proposition advanced by the author that the best method of determining the proportions of concrete is, first, to decide upon the ratio of sand to cement in the mortar, and, second, to determine the ratio of mortar to stone, is undoubtedly correct, although the conventional method of statement of the relative volumes of the various ingredients is extremely convenient and will doubtless continue to be used.

The paper treats the subject apparently only from the standpoint of the crushing strength of the concrete produced. In very large masses and in foundations the crushing strength is, perhaps, the most important quality of concrete; but concrete is used for many purposes where other properties than crushing strength are of very great and, in fact, of controlling importance. Thus, in exposed positions, a concrete which will resist the action of frost is of the first importance. In other cases a water-tight concrete is required, and in still others it is desirable to have concrete which will completely fill forms and yield smooth surfaces, and is of such consistency as to allow the upper surface to be troweled. For all of these purposes a concrete as free from voids as possible is desirable; and to secure satisfactory results it is necessary to use an amount of mortar somewhat more than sufficient to completely fill the voids in the stone.

The proportions of voids in broken stone, gravel and sand are often very large. They are frequently underestimated by defective methods of measurement. The most obvious, and, perhaps, the most common, method used to determine voids is to place the material in a receptacle of known capacity, and to pour as much water into it as possible, measuring the volume of water used. This procedure invariably gives results which are too low, the amount of error depending upon several conditions. The sand or gravel for examination is rarely perfectly dry. Two or 3% of water by weight do not make a material appear particularly moist; but 2 or 3% of water by weight are equal to from 3 to 5% by volume, and if the voids are measured by displacement the results will be too low by this amount.

Another and oftentimes a more serious source of error arises from the fact that all the air is not displaced by the water. With coarse, clean gravels the air can be displaced almost completely, and the error from this source is not large. With dirty materials the error is much greater, and with sands the air cannot be even approximately displaced in this way.

The most accurate and convenient method of determining the voids, know to the writer, is to determine, first, the specific gravity of the solid particles, and, second, the specific gravity of the material as a whole, including the voids. The percentage of space occupied by the particles is then obtained by dividing the specific gravity of the mass as a whole, including the voids, by the specific gravity of the

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solid particles, and the voids are represented by the difference between the whole volume and the space occupied by the solid particles.

In case the material used for this purpose is not absolutely free from water, the whole or a part of it must be dried after the experiment to determine the amount of water in it, and a correction applied therefor. It will not do to dry a moist material before taking its specific gravity, as the closeness of packing will be very likely to be different for wet and dry materials.

It is usually a comparatively easy matter to determine the specific gravity of the solid particles, but for large classes of materials the specific gravities are so uniform that the errors introduced by assuming an average value of 2.65 are very slight. The specific gravities of sands and gravels of the glacial drift of the Eastern States and of all silicious sands are very close to this figure, and much of the broken stone used for concrete is of nearly the same density. The proportion of voids is thus a direct function of the weight per cubic foot, after allowance is made for moisture. Loosely placed stone or sand weighing 83 lbs. per cubic foot has 50% of voids. When compacted to 45% of voids, the weight is increased to 91 lbs. per cubic foot. In the speaker's experience this represents approximately the condition of ordinary building sand as it is usually measured for concrete, and corresponds closely with the weight of sand mentioned by the author. When further compacted to 40% of voids the weight is 99 lbs. per cubic foot. With 35% of voids the weight is 108 lbs., and with 30% of voids, 116 pounds per cubic foot.

The voids in clean gravels or in broken stone without special packing are seldom less than 40%, although they are less when sand, small fragments of stone or other fine materials are present in considerable amount in the interstices of the larger material. When ballast for concrete contains such material the amount of sand in the mixture should be reduced by a corresponding amount, but no reduction should be made in the quantity of cement. That is, the fine material in the ballast should be computed as sand, and as entering into the composition of the mortar and not as a part of the ballast.

The proportions of mixing used by the author, namely, with mortar equal to 33 and 40% of the volume of the broken stone, would seem likely to result in the production of concrete with a large proportion of voids. The speaker has never had occasion to use a broken stone which would yield a concrete entirely free from voids when used with as little as 40% of its volume of mortar, and he would expect with as low as 33% that the voids would be large and conspicuous. That this was the case in the author's experiments is suggested by the fact that the percentage of shrinkage on ramming, given in Table No. 3, is substantially the same for 33% of mortar

Mr. Hazen. and for 40% of mortar. If the 33% of mortar had filled the voids there would have been an excess of mortar when 40% was used, and the shrinkage by ramming would have been less. As the shrinkage was as much with 40% of mortar as it was with 33%, it seems almost certain that with the smaller quantity the voids were not entirely filled; and in view of general experience, the speaker would be inclined to question whether even the 40% was adequate to fill all the voids.

There are three possible conditions for the amount of mortar in concrete. First, the mortar may be insufficient to fill the voids between the stones; second, it may just fill them; or, third, there may be an excess of mortar. In the first condition, with the voids not entirely filled with mortar, the stones will come to a direct bearing upon each other, and if there is sufficient mortar to prevent them from slipping upon each other, the crushing strength of the concrete may be as great as if more mortar had been used; and for certain purposes such concrete is entirely satisfactory. If the mortar exactly fills the voids the strength certainly will not be less, and for many purposes the concrete will be more satisfactory. Owing to the impossibility of securing perfect mixing, it is practically necessary to use more than enough mortar to fill the voids, in order to make certain that the voids will be filled at every point; and this is the reason usually given for the use of more mortar than the equivalent of the voids, but it is also possible that for some purposes an excess of mortar in itself is desirable.

The use of a sufficient amount of cement to fill the voids in the sand is probably desirable for some purposes, but it is often less necessary than the filling of the voids in the gravel with mortar. Thus, a mortar composed of one part of Portland cement and three parts of sand is very strong and durable, and sufficient for many purposes, although it can hardly be considered that this amount of cement will entirely fill the voids in the sand. The remaining voids, however, are so small that the mortar is nearly impervious to water, and concrete made from it takes substantially as smooth a finish as that made from a richer mortar.

It should be remembered also that it is not possible to make a concrete entirely free from voids. This arises from the fact that concrete, to be handled conveniently and satisfactorily, requires to have a certain portion of water in it. Some of this water combines with the cement in the process of setting. In some experiments made by the speaker the proportion of water combining in this way amounts to about 8% of the weight of the cement, or to about 1% of the weight of the concrete. This quantity of water is entirely inadequate to give the concrete the proper consistency for the purpose of placing and ramming. In some experiments made by the speaker the proportion of water in concrete as used in actual work amounted 3 or 4% of the

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weight of the concrete as placed, in addition to the amount required for combination with the cement. To mix the concrete much dryer than this requires a great deal more ramming, which is apt to be neglected under other than very severe inspection, and it is impossible to get smooth, finished surfaces with concrete that is too dry. The above-mentioned percentage of water amounts to 8 or 10% by volume, exclusive of the water which goes into combination with the cement. After the concrete has set, this water will evaporate in the course of time, if it has an opportunity, and the resulting concrete will have a corresponding volume of voids. With the method of manufacture commonly used it is not easy to see how these voids can be entirely avoided, but it is perfectly practicable to have all the voids of extremely small size and in the mortar, and to have no voids visible to the eye and resulting from an insufficiency of mortar in the ballast. Practically, the best concrete, under ordinary conditions of mixing and ramming, and with gravel or broken stone having a specific gravity of about 2.65, will rarely weigh more than 150 lbs. per cubic foot, which corresponds to 9% of voids. Concrete of this compactness, however, with proper materials and well set, resists very well the action of the weather, and the voids are not usually apparent to the eye, or sufficient to allow water to percolate through them to an important extent.

There is a matter in connection with the proportions of mixing concrete which perhaps gives rise to even more perplexity than those mentioned by the author, namely, the measurement of the volume of the cement. The volume of a barrel of cement may be taken as that of a barrel when full, or as the actual volume of the cement in the barrel after it has been shaken down by transportation to a somewhat smaller volume, or the cement may be turned out and measured in another receptacle. The speaker considers that the ideal condition is to mix concrete by weight, and as an approach to this condition in his own practice he has often made the arbitrary assumption that a barrel of cement weighing 380 lbs. net, occupies a space of 3.8 cu. ft., making the weight of cement 100 lbs. per cubic foot. The weight of the cement is determined or checked, no attention is paid to its actual volume, but for the purpose of computation one cubic foot is assumed for each 100 lbs. of cement. The average weight of broken stone or gravel is also approximately 100 lbs. per cubic foot, and on this basis the volumes also express the weights of these materials. The sand is usually lighter, but after the actual weight per cubic foot is determined, the approximate weight can be determined from the volume.

In computing the amounts of the various materials required to make a cubic yard of concrete the following procedure has been used by the speaker with satisfactory results. The weight of a barrel of cement is taken, and 8% is added for water of combination. To this is added the weight of sand free from water which will be used with a

Mr. Hazen. barrel of cement, and the corresponding weight of ballast. An estimate is made of the weight per cubic foot of the finished concrete, which, for compact work, may be taken at 150 lbs. per cubic foot, and for concrete with more ballast in proportion to sand, at a lower figure, which can be determined by experiment or estimated approximately from the appearance of the finished work. The ratio of the sum of the weights of these materials to the assumed or determined weight of a cubic foot of finished concrete will give the number of cubic feet made from a barrel of cement. The quantities of the various materials required to make a cubic yard of concrete can then be readily computed. Thus, in a recent case, concrete was mixed in the proportions of one part of cement, computed as above, three parts of sand weighing 90 lbs. per cubic foot, and five parts of broken stone and gravel weighing 100 lbs. per cubic foot. The concrete when dry weighed about 147 lbs. per cubic foot. The proportions of mixing differed slightly in different parts of the work, but the above proportions are about the average.

The computed weight of concrete made from a barrel of cement was as follows:

One barrel of cement	380 lbs.
Water of combination, 8%	30 "
Sand, 11.4 (3.8 x 3) cu. ft., at 90 lbs., dry	1 026 "
Gravel, 19 (3.8 x 5) cu. ft., at 100 lbs., dry	1 900 "
Total weight of concrete made from 1 barrel of cement	3 336 "

Volume produced, $3\,336 \div 1.47 = 22.7$ cu. ft.

Cement required for 1 cu. yd., $27 \div 22.7 = 1.19$ barrels.

From the records of the work 15 085 cu. yds. of concrete were made from 18 584 barrels of cement, or 1.23 barrels of cement per cubic yard, no deduction being made for ordinary waste, nor for concrete outside of contract lines.

This concrete contains very few voids visible to the eye, when broken or as exposed where it has been placed against forms. All top surfaces were troweled smooth before setting and the proportions were such as to allow this to be satisfactorily done. The proportion of sand and gravel necessary to secure this result was determined by experiment, and was the lowest which could be satisfactorily used. With the volume of sand equal to half of that of the ballast, there were too many voids exposed when the forms were taken down, and there was difficulty in making the top surfaces smooth by troweling. It was thus necessary to use a volume of sand equal to 60% of the volume of the ballast, but for other purposes a wider mixture would be satisfactory and cheaper.

B. M. Harrod, Past-President, Am. Soc. C. E. (by letter).—This paper is very practical and useful. The tables give exact information concerning the losses in bulk of the several ingredients, when made into concrete, with which those who have used that material have been familiar in a general way. Five units of cement give about four units of mortar. This is all absorbed in the voids of the stone, which also loses about 10% of its bulk by the readjustment of its particles under the rammer. Mr. Harrod.

In the writer's note-book there is a very old memorandum, to the following effect:

"The theory of concrete-making is to take a barrel of shells, and pour in as much water as the interstices will admit. Then put in a quantity of mortar equal to the bulk of water admitted. If you make any deviation from this, it will be by increasing the mortar."

The authority for this formula is the late General Joseph G. Totten, Chief of Engineers, U. S. A., Hon. M. Am. Soc. C. E.

Adopting this theory, good results can always be assured, whenever there is any doubt concerning the proportion of voids in the stone, as there often is, by specifying a test, with a tight box, containing about half a cubic yard. When this is filled with loose stone, sufficient water should be poured in to fill the voids. After the stone and the sides of the box are saturated and the water is withdrawn, a second charge of water should be poured in and carefully measured. The mass of mortar used should exceed that of the water by 5 or 10 per cent.

The shells alluded to in General Totten's formula are largely used in New Orleans in the place of broken stone on account of their fitness and cheapness. They cost from $1 to $1.25 per cubic yard, which is about half the cost of good broken stone or clean gravel. They are the shells of a small clam (*gnathodon cuneatus*) about the size of a half dollar. They are found in banks, washed up along the shores of the brackish bayous and lakes in the vicinity of New Orleans, and also in kitchen middens, in such quantities that they are dredged up free from any impurity except a little sand. They have been largely used for roads for many years, and the end of the supply is not in sight. They make a solid concrete, as the thickness and material of the shell give it ample strength, and its concavity forms a good bond. They require a little more mortar than broken stone, and the mixing must be very thoroughly done. The weight of shell concrete is 137.8 lbs. per cubic foot, which is less than that of concrete made with broken stone. This fact, in some cases, is of importance.

William M. Hall, M. Am. Soc. C. E. (by letter).—The writer readily admits that the new nomenclature which Mr. Rafter suggests is very good, but can see no reason why the old nomenclature should obscure from those who are looking for it the true theory of concrete mixtures. Mr. Hall.

Referring to the amount of mortar required for mixing with the

Mr. Hall. aggregate, the author well says, "there being, indeed, very little necessary except to insure the filling of the voids in the aggregate material with mortar." It appears well, also, to emphasize the fact that the same statement is true in reference to filling the voids in the sand with cement; also, in reference to the volume of the voids in ordinary sands and aggregates; and, also, that where the sand, cement and aggregate are accurately proportioned, with reference to voids, unless they are thoroughly mixed, the concrete may be, in places, almost worthless.

In lock and dam construction in Kentucky, the writer found voids in sand and aggregate as follows:

The average voids in sand was 31% of the volume.

The sand was clean and sharp, and the size of grains as follows:

Held by No. 20 sieve, 11 per cent.

Passed " 20 " and held by No. 30, 14 per cent.

" " 30 " " " 50, 53 "

" " 50 " 22 per cent.

The voids in crushed stone and in gravel and mixtures of the two were as follows:

100%	of crushed	$2\frac{1}{2}$-in.	stone;	voids,					48%	of volume.
80%	"	$2\frac{1}{2}$ "	"	20%	$1\frac{1}{2}$-in.	gravel,	voids		44%	"
70%	"	$2\frac{1}{2}$ "	"	30%	$1\frac{1}{2}$ "	"	"	"	41%	"
60%	"	$2\frac{1}{2}$ "	"	40%	$1\frac{1}{2}$ "	"	"	"	$38\frac{1}{2}$%	"
50%	"	$2\frac{1}{2}$ "	"	50%	$1\frac{1}{2}$ "	"	"	"	36%	"
				100%	$1\frac{1}{2}$ "	"	"	"	35%	"

The above results are the averages of a number of tests made from several different barges of material. However, the volume of voids in the various mixtures ran quite uniform. The sand and gravel were very clean Ohio River products, taken from the shoal at the mouth of Green River, Ky. The stone was crushed Green River blue limestone, and in texture and color was much like the Hudson River trap. The stone passed a $2\frac{1}{2}$-in. screen and also a fine one to clean it of dust. The gravel passed a $1\frac{1}{2}$-in. screen and then a fine one to remove the sand.

In sea-coast fortification construction in Massachusetts during the season of 1898, the writer found the volume of voids in sand and aggregate as follows:

The average of voids in sand was $31\frac{2}{10}$% of the volume.

The voids in stone and gravel and mixtures of the two were as follows:

100%	of crushed	$2\frac{1}{2}$-in.	trap;	voids,					50%	of volume.
60%	"	$2\frac{1}{2}$ "	"	and	40%	$1\frac{1}{2}$-in.	gravel,	voids	$38\frac{1}{4}$%	of volume.
50%	"	$2\frac{1}{2}$ "	"	"	50%	$1\frac{1}{2}$ "	"	"	36%	of volume.
					100%	$1\frac{1}{2}$ "	"	"	35%	of volume.

The stone was Hudson River trap with the dust removed. The sand and gravel were taken from Long Island Sound, near Willets Point. The gravel was clean, but the sand contained about 8% of loam. Mr. Hall.

By the foregoing, the writer wishes to call attention to the saving which may be effected by mixing gravel with stone, as an aggregate, instead of using stone alone. He has failed to find similar records in the literature on the subject; but as the voids in these two stones, and the corresponding mixtures with gravel are so nearly uniform, he thinks that nearly the same results will be found with other hard stones having lines of fracture similar to these.

The comparison which the author makes between the concretes made with dry, plastic and very wet mortars are especially interesting. They would be much more instructive, however, if they had included a series of cubes which had seasoned for three or four years.

The conclusions likely to be drawn from these tests are, that dry concrete, when set and hardened, is stronger than wet. It is well known that this result may be obtained from 1-in. mortar briquettes a few months old, and the reverse result from those several years old. Moreover, in tamping the test pieces, it may be assumed that all kinds of concrete were equally well tamped; whereas, in actual practice, the obtaining of well-tamped work with dry concrete is much more difficult than with plastic or wet concrete. In actual construction, the writer prefers concrete with plastic or slightly wet mortar to that which is dry. After setting has commenced, he believes it is advantageous to apply water to the outside surface, or to the interior of the concrete mass by means of well-holes.

Ira O. Baker, M. Am. Soc. C. E. (by letter).—The author deserves credit for the thorough investigation given the practical problem presented, and the special thanks of the profession for the full and careful presentation of the data, particularly the valuable summaries and comparisons. Mr. Baker.

The writer is much interested in the effect of the amount of mortar upon the strength of the concrete. Half of the specimens tested contained mortar equal to 33% of the broken stone, and half an amount equal to 40 per cent. Apparently, the broken stone contained 43.3% of voids before tamping and 37.4% after tamping,* and the mortar was measured without tamping. Therefore, owing to the decrease of the volume of the mortar in ramming the concrete, and to the effect of moisture in increasing the volume of the sand, it is probable that the 40% of mortar was not much, if any, more than equal to the voids in the broken stone, when both were rammed. Under the same assumption, the 33% of mortar was equal to about 80% of the voids in the broken stone after the concrete had been tamped into place.

* Report of the New York State Engineer and Surveyor for the year ending September 30th, 1894.

Mr. Baker. Whether or not the 40% of mortar filled the voids, one would naturally expect that an addition of 25% of mortar would give, at least approximately, 25% additional strength. The results of the experiments appear not to support this conclusion.

In Table No. 5 the results are summarized according to the plasticity of the mortar, from which it appears that concrete con-

TABLE No. 5.

Plasticity of mortar.	Amount of Mortar.		Strength of the 40% concrete in terms of that of the 33%.
	33%	40%	
Dry	2 408	2 532	105%
Plastic	2 259	2 329	103%
Excess	2 133	2 227	104%
Mean	2 267	2 363	104%

taining 40% of mortar is only 4% stronger than that containing 33 per cent. Apparently, the plasticity of the mortar makes no material difference in the result.

Summarizing the results according to the proportions of the mortar gives Table No. 6, from which it appears that the concrete containing 40% of mortar is only 3% stronger than that containing 33 per cent.

TABLE No. 6.

Proportions of the mortar.	Amount of Mortar.		Strength of the 40% concrete in terms of that of the 33%.
	33%	40%	
1 : 2	2 640	2 820	107%
1 : 3	1 893	1 905	102%
1 : 4	1 684	1 689	100%
Mean	2 072	2 138	103%

The mean increase in strength, from Tables Nos. 5 and 6 differs, owing partly to omitted decimals and partly because the two tables do not refer to exactly the same experiments. The writer, to save labor, used the summaries in the original paper and they were prepared for a different purpose.

The most interesting deduction from Table No. 6 is that the strength of the concrete increases with the richness of the mortar; and

therefore it is better to increase the proportion of the cement in the mortar than to increase the proportion of mortar in the concrete. This is doubtless due to the greater ease with which the richer mortar will flow into the voids during ramming. Mr. Baker.

Summarizing the results according to the method of storage gives Table No. 7, from which it appears that the method of storage is unimportant. Figured in this way, the concrete with 40% of mortar is 2% stronger than that with 33% of mortar.

TABLE No. 7.

Method of storage for first four months.	Amount of Mortar.		Strength of the 40% concrete in terms of that of the 33%.
	33%	40%	
Under water	2 424	2 448	101%
In a cool cellar	1 992	2 009	101%
In an open shed	1 932	1 999	104%
Under wet burlap	2 007	2 096	104%
Mean	2 089	2 138	102%

Summarizing the results according to the kind of cement used, Table No. 8 is obtained. It is remarkable that the excess strength

TABLE No. 8.

Brand of Portland cement.	Amount of Mortar.		Strength of the 40% concrete in terms of that of the 33%.
	33%	40%	
Genesee	2 587	2 344	91%
Wayland	2 425	2 441	101%
Iron Clad	2 142	2 295	115%
Empire	1 970	1 955	107%
Champion	2 206	2 214	91%
Mean	2 266	2 256	101%

for the concrete containing 40% of mortar, differs so much for the different cements.

It is hoped that the author will explain why the greater percentage of mortar adds so little to the strength of the concrete, and also why this increase varies so greatly for the different cements. The writer is unable to get any light on these questions from the published data. The experiments are so extensive and appear to have been conducted so carefully, that it does not seem reasonable to assume that variations therein are chargeable with the above remarkable results.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The author's criticism on the present practice of making concrete will be concurred in by every engineer who has given the matter any study. It is a humiliating spectacle to see first-class material, manipulated in an irrational and wasteful manner, developing results which cannot be shown to possess a single redeeming feature.

The tabular results submitted by the author are full of interesting matter, and bring out prominently one feature of considerable practical importance which is not fully appreciated by engineers. The crushing strength of ordinary cement mortars diminishes very nearly in proportion to the quantity of sand added; whereas the author's tables show conclusively that, in the case of concrete, this general law is materially modified. The same general law of diminution of strength follows until the theoretic mortar, 3 to 1, is reached; but for some reason, in the case of all the lower grades of concrete blocks, made with 4 to 1, 5 to 1 and 6 to 1 mortars, the corresponding diminution in crushing strength is noticeable, but much less in degree than would naturally be expected.

Taken as a class, the writer thinks the lower grade concretes are very much neglected by engineers. In most cases where concrete is used the unit-strength of the concrete is not called into play at all. Dead-weight and the distribution of pressures are all that are required, and the lower-grade concretes fulfill these conditions completely and economically.

It is to be regretted that the author did not state clearly the age of the concrete tested, the footnote leading one to suppose that the blocks were about 12 months old.

As to the relative merits of dry, plastic and excess mortars, the writer thinks there is much more danger to be expected from the lack of water than from an excess.

Mr. Von Schon. Mr. H. Von Schon (by letter).—During the spring and summer of 1898 investigations of concrete were carried on under the writer's direction at Sault Ste. Marie, Mich., for the purpose of establishing the most satisfactory and economic constituents and mixtures of concrete to be used in the substructure of a hydraulic power house. The structure in question, being partly subaqueous, must needs be of masonry, and as the only available native material, Potsdam sandstone, was considered of insufficient homogeneity, concrete was selected. The character of the construction called for materials of great density and strength, and the climatic condition for resistance to low temperature, while the extent of the work, requiring about 50 000 cu. yds. of concrete, suggested a diligent search for the most economical mixture which would secure the desired result. It was therefore to determine the kind of cement and aggregate, and the ratio of mixtures which,

together, would yield the proper material for the purpose in hand, that these tests were entered upon. Mr. Von Schon.

The general programme as outlined for these tests was:

First.—To operate practically, that is, to avoid the introduction of refinements which could not be followed in actual construction.

Second.—To establish the characteristics of all materials used, namely, the analysis and normals of the cements; the weight, voids, and fineness of the sand; and the weight, voids, sizes and absorption of materials to be used as aggregates.

Third.—To make two concrete bars from each of the following combinations: Four different kinds of cement, four different materials for aggregates and four different ratios of mixtures.

Fourth.—To test each bar to the breaking point, when 60 days old, by applying a load at the center.

All the manipulations were entrusted to two men who had had the advantage of several years' training in the United States cement testing laboratory at this point, under L. C. Sabin, Assoc. M. Am. Soc. C. E., U. S. Assistant Engineer, and Mr. W. W. Dann, Assistant Engineer, who had direct supervision and kept the records. A building was conveniently arranged with material bins, testing and mixing rooms. Storage for the manufactured bars was provided for outside the building.

TABLE No. 9.—Characteristics of Cements.

		Class *E.*	Class *R.*	Class *F.*	Class *N.*
Analysis.	Silica	23.08	21.70	27.48	23.16
	Alumina	5.69	8.76	.....	6 33
	Iron oxide	5.35	1.27	14.42	1.71
	Lime	62.38	63.55	50.39	36.08
	Magnesia	1.21	2.96	3.18	20.38
	Potash and soda	1.66	1.12	....	5.26
	Carbonic acid, etc	0.60	0.50	4.36	7.07
Weight per cubic foot in pounds		103.08	98.45	77.70	62.78
Fineness passing No. 100 sieve, per cent.		91.4	90	99.00	.....
" " No. 200 " "		73.05	71	95.00	.....
Initial set		1 h. 35 m.	2 h. 25 m.	1 h. 35 m.	0 h. 30 m.
Hard set		3 h. 10 m.	4 h. 40 m.	2 h. 30 m.	1 h. 00 m.
Tensile strength 1-3		Lbs.	Lbs.	Lbs.	Lbs.
7 days		235	243	170	32
28 days		340	322	243	67

Four different kinds of cement, representing the chief raw materials from which American cements are manufactured (calcareous rock, calcareous earth, natural cement rock and furnace slag), and each of them a cement of high standing among its class, were used in these tests. The sand was procured from the bed of St. Mary's River,

Mr. Von Schon.

TABLE No. 10.—Characteristics of Sand and Aggregates.

	Sand.	Sand-stone.	Boulder stone.	Gravel.	Furnace slag.
Size—					
Passing 1½-in. ring, per cent.....	.....	100.00	100.00	100.00	.....
Retained on 1-in. ring, per cent....	.....	100.00	100.00	10.70	.....
" " ¾-in. ring, "	.....	.....	.....	23.65	1.10
" " No. 4 sieve, per cent.	.....	.....	.....	8.70	2.86
" " " 10 " "	.....	.....	.....	17.14	45.62
" " " 20 " "	4.17	.....	.....	21.76	36.92
" " " 30 " "	12.52	.....	.....	6.49	8.26
" " " 40 " "	44.44	.....	.....	5.96	3.24
Passing " 40 " "	38.87	.....	.....	5.59	2.00
Weight per cubic foot dry, pounds....	96.45	79.32	84.00	107.36	126.23
Weight per cubic foot wet, pounds....	113.46	82.62	86.70	.....	.....
Voids in per cent. of volume.....	41.7	45.3	48.7	34.08	43.8

TABLE No. 11.—Relative Volumes of Constituents in One Cubic Foot of Concrete.

Using Sand Stone for Aggregate.

Materials.	Mixture *A*.			*B*.			*C*.			*D*.			*E*.		
	Cubic feet.	Pounds.	Ratio of volume.	Cubic feet.	Pounds.	Ratio of volume.	Cubic feet.	Pounds.	Ratio of volume.	Cubic feet.	Pounds.	Ratio of volume.	Cubic feet.	Pounds.	Ratio of volume.
Cement—															
Class *E*.....	0.19	19.5	1.00	0.21	21.5	1.00	0.23	23.7	1.00	0.25	25.9	1.00	0.19	19.5	1.0
" *R*.....	0.19	18.6	1.00	0.21	20.6	1.00	0.23	22.6	1.00	0.25	24.7	1.00	0.19	18.6	1.0
" *F*.....	0.19	14.7	1.00	0.21	15.9	1.00	0.23	17.8	1.00	0.25	19.5	1.00	0.19	14.7	1.0
" *N*.....	0.19	11.9	1.00	0.21	13.1	1.00	0.23	14.3	1.00	0.25	15.7	1.00	0.19	11.9	1.0
Lime.....	....	....	....	....	....	....	....	....	....	....	....	....	0.06	3.8	0.3
Sand.....	0.45	43.7	2.4	0.50	48.3	2.4	0.55	53.4	2.4	0.60	58.2	2.4	0.60	58.2	3.1
Water.....	0.11	7.0	0.6	0.13	8.0	0.6	0.14	9.0	0.6	0.16	10.0	0.6	0.19	12.0	1.0
Sand Stone (S. S.)....	1.0	82.6	5.3	1.0	82.6	4.8	1.00	82.6	4.4	1.00	82.6	4.0	1.00	82.6	5.3

Using Boulder Stone for Aggregate.

Cement—															
Class *E*.....	0.20	21.0	1.00	0.22	23.0	1.00	0.24	25.3	1.00	0.26	27.4	1.00	0.20	21.0	1.0
" *R*.....	0.20	20.0	1.00	0.22	22.6	1.00	0.24	24.1	1.00	0.26	26.1	1.00	0.20	20.0	1.0
" *F*.....	0.20	15.7	1.00	0.22	17.4	1.00	0.24	19.0	1.00	0.26	20.6	1.00	0.20	15.7	1.0
" *N*.....	0.20	12.7	1.00	0.22	14.0	1.00	0.24	15.4	1.00	0.26	16.7	1.00	0.20	12.7	1.0
Sand.....	0.49	47.0	2.45	0.54	52.0	2.45	0.59	56.6	2.45	0.64	61.4	2.46	0.64	61.4	3.0
Water.....	0.13	8.0	0.65	0.14	9.0	0.65	0.15	10.0	0.65	0.16	10.6	0.65	0.20	12.5	1.3
Lime.....	....	....	....	....	....	....	....	....	....	....	....	....	0.06	3.8	0.2
Boulder Stone (B. S.).	1.0	86.7	5.0	1.0	86.7	4.35	1.0	86.7	4.16	1.0	86.7	3.85	1.0	86.7	5.0

Lime weighs 63.75 lbs. per cubic foot.

and is known as Point aux Pins sand. The materials used for aggregates were of the native Potsdam sandstone, granitic boulder stone, gravel and furnace slag (see Tables Nos. 9 and 10). Mr. Von Schon.

The ratios used were: For the "*A*" mixture, such proportions as would just fill the voids, and for "*B*," "*C*" and "*D*" mixtures of a respective addition of 5, 10 and 15% of mortar to the "*A*" mixture. A few bars were made of mixture "*E*" having the ratio of "*D*" mixture, but substituting lime for the cement in the excess mortar quantity. Table No. 11 gives the quantities and ratios of materials for each of the above mixtures, yielding theoretically 1 cu. ft. of concrete.

Knock-down forms were constructed of thoroughly seasoned timber lined with tin, the inside dimensions being precisely 6 ins. square and about 24 ins. long. The mixing was done in batches of 1 cu. ft. of concrete from which two forms were filled. The operations were the customary ones, namely, spreading dry sand on clean platforms, adding the cement, mixing the sand and cement dry, adding water, mixing the mortar, spreading the wet aggregate on a second platform, adding mortar to the aggregate, mixing the concrete, filling the concrete in the form in 6-in. layers and ramming.

The mixing tools used were spades and hoes. Water was used from the city supply at its natural temperature, about 50° Fahr.; the quantity was determined by the condition of the mortar which was kept just plastic, without any water standing on or running from it. The rammers used for tamping were of oak, weighing about 8 lbs. The weight of all materials used, of the forms before filling and of the excess material after filling, was taken accurately and recorded. All bars were made during April and May, 1898, and broken at an age of 60 days, during June and July, 1898.

After filling the forms, the end of each bar was marked by its serial number, weighed and left undisturbed for from 2 to 8 days, when the form was removed and the bar placed in storage. At the age of 60 days each bar was weighed and then broken. The breaking apparatus consisted of two rigid timber frames 18 ins. apart, and of a scale platform suspended from a steel beam of triangular shape. The bar was adjusted on the frame, the scale beam with knife-edge on the center of the bar, and the platform was loaded with weights, and finally by pouring sand, until the bar broke, when all weights were taken and recorded. The fractures of the broken bars were examined, and those of exceptional appearance were photographed.

Some of the broken bars were kept for a year, during which time they were exposed to the prevailing weather of the summer and winter season, the latter being especially severe and long. They were then re-examined, weighed, placed in water of natural temperature and weighed daily until no additional absorption was observable (see Table No. 13.)

Mr. Von Schon. TABLE No. 12.—History of Fabricated Bars and Breaking Tests.

Fabricating Data.					Test Data.				
No. of bar.	Cement.	Aggregate.	Mixture.	Weight of bar.	Days in Form.	Days in Air.	Weight of bar before breaking.	Breaking weight.	Character of break.
1	2	3	4	5	6	7	8	9	10
1..	E	S. S.	A	73.5	2	58	71.0	1122.5	Clean and sharp.
2..	E	S. S.	A	73.5	2	58	71.0	1380.75	" "
9..	E	S. S.	B	73.5	5	55	71.5	1798.75	" "
10..	E	S. S.	B	76.0	5	55	72.0	1671.0	" "
17..	E	S. S.	C	75.0	3	57	72.25	2302.25	" "
18..	E	S. S.	C	75.25	3	57	72.5	2179.00	" "
25..	E	S. S.	D	77.75	5	55	75.25	2634.25	" "
26..	E	S. S.	D	78.25	5	55	75.75	2567.75	" "
33..	E	S. S.	E	73.25	6	54	69.25	862.00	" "
34..	E	S. S.	E	73.25	6	54	69.75	777.25	" "
3..	R	S. S.	A	76.25	2	58	72.5	1444.25	" "
4..	R	S. S.	A	72.25	2	58	69.5	1264.00	" "
11..	R	S. S.	B	75.5	5	55	73.5	1461.00	" "
12..	R	S. S.	B	75.0	5	55	73.0	1339.75	" "
19..	R	S. S.	C	76.75	5	55	74.5	2130.00	" "
20..	R	S. S.	C	76.5	5	55	74.5	2064.5	" "
27..	R	S. S.	D	75.5	5	55	72.5	2307.00	" "
28..	R	S. S.	D	77.75	5	55	75.0	2027.75	" "
35..	R	S. S.	E	72.5	6	54	69.0	1558.5	" "
36..	R	S. S.	E	74.5	6	54	71.0	1357.0	" "
5..	F	S. S.	A	72.25	5	55	69.75	821.75	Rough and crumbly
6..	F	S. S.	A	74.5	5	55	71.5	511.5	" "
13..	F	S. S.	B	75.25	3	57	72.5	903.75	Clean.
14..	F	S. S.	B	75.00	3	57	72.25	993.5	"
21..	F	S. S.	C	73.00	5	55	71.00	1153.00	Clean but green.
22..	F	S. S.	C	76.75	5	55	74.75	1373.00	" "
29..	F	S. S.	D	77.00	6	54	74.5	962.25	" "
30..	F	S. S.	D	77.00	6	54	74.5	919.75	" "
37..	F	S. S.	E	73.25	6	54	71.0	1013.25	" "
38..	F	S. S.	E	75.25	6	54	73.0	1034.50	" "
7..	N	S. S.	A	67.0	5	55	Broke in handling		
8..	N	S. S.	A	69.75	5	55	"	"	
15..	N	S. S.	B	73.75	8	52	69.0	285.5	Crumbled.
16..	N	S. S.	B	73.75	8	52	70.25	305.75	"
23..	N	S. S.	C	73.5	5	55	69.0	284.75	"
24..	N	S. S.	C	73.5	5	55	69.0	229.50	"
31..	N	S. S.	D	73.75	6	54	69.0	351.25	"
32..	N	S. S.	D	74.00	6	54	69.75	281.00	"
39..	N	S. S.	E	74.00	10	50	69.00	267.00	"
40..	N	S. S.	E	72.75	10	50	67.5	246.5	"
41..	E	B. S.	A	79.25	4	56	77.25	2386.00	Clean and sharp.
42..	E	B. S.	A	79.00	4	56	77.25	2829.75	" "
49..	E	B. S.	B	77.25	22	38	75.5	2391.5	" "
50..	E	B. S.	B	80.25	22	33	78.75	2861.00	" "
57..	E	B. S.	C	80.25	18	42	78.75	2849.25	" "
58..	E	B. S.	C	80.25	18	42	78.5	3120.75	" "
65..	E	B. S.	D	81.75	5	55	80.0	3361.75	" "
66..	E	B. S.	D	79.75	5	55	77.75	3200.5	" "
73..	E	B. S.	E	78.00	2	58	75.5	2480.0	" "
74..	E	B. S.	E	78.00	2	58	75.25	2800.75	" "
43..	R	B. S.	A	79.75	4	56	77.5	3119.00	" "
44..	R	B. S.	A	79.75	4	56	77.5	2436.00	" "
51..	R	B. S.	B	81.25	22	38	79.5	3121.75	" "
52..	R	B. S.	B	79.5	22	38	77.5	3386.75	" "
59..	R	B. S.	C	80.0	18	42	78.75	2988.00	" "
60..	R	B. S.	C	80.0	18	42	78.50	3282.50	" "
67..	R	B. S.	D	80.0	5	55	77.75	3289.50	" "
68..	R	B. S.	D	80.5	5	55	78.25	3004.25	" "
75..	R	B. S.	E	78.25	2	58	74.00	2499.25	" "
76..	R	B. S.	E	79.25	2	58	76.50	2653.00	" "

TABLE No. 12.—(*Continued.*)

1	2	3	4	5	6	7	8	9	10
45..	*F*	B. S.	*A*	80.00	4	56	77.5	1183.25	Clean but green.
46..	*F*	B. S.	*A*	73.00	4	56	70.75	901.00	" "
53..	*F*	B. S.	*B*	81.00	18	42	79.25	1266.75	" "
54..	*F*	B. S.	*B*	79.75	18	42	78.00	1490.25	" "
61..	*F*	B S.	*C*	80.25	6	54	78.5	1163.75	" "
62..	*F*	B. S.	*C*	78.50	6	54	77.0	1456.50	" "
69..	*F*	B. S.	*D*	77.75	5	55	75.5	1078.00	" "
70..	*F*	B. S.	*D*	79.50	5	55	77.25	978.5	" "
77..	*F*	B. S.	*E*	78.00	4	56	75.75	956.25	" "
78..	*F*	B. S.	*E*	78.25	4	56	75.75	935.25	" "
47..	*N*	B. S.	*A*	75.75	22	38	71.5	364.25	Crumbled.
48..	*N*	B. S.	*A*	79.75	22	38	75.25	414.5	"
55..	*N*	B. S.	*B*	77.25	18	42	72.25	267.0	"
56..	*N*	B. S.	*B*	77.00	18	42	70.5	560.0	"
63..	*N*	B. S	*C*	77.75	9	51	72.75	596.0	"
64..	*N*	B. S.	*C*	77.25	9	51	72.25	422.5	"
71..	*N*	B. S.	*D*	75.0	8	52	71.0	203.0	"
72..	*N*	B. S.	*D*	77.0	8	52	72.0	318.75	"
79..	*N*	B. S.	*E*	77.25	4	56	73.5	486.0	"
80..	*N*	B. S.	*E*	77.75	4	56	73.0	419.5	"

NOTE.—An excess of variable quantity remained over after the making of each pair of bars.

TABLE No. 13.—ABSORPTION TESTS.

No. of bar.	Age, in days.	Weight of bar.	Condition before absorption.	Weight, in Pounds after being Submerged.	
				1 day.	2 days.
1.....	381	71	hard.	74.0	74.0
9.....	381	50	"	51.5	51.5
11.....	381	73	"	75.5	75.5
17.....	379	39	"	40.0	40.0
19.....	376	74	"	76.0	76.0
27.....	376	72	"	73.5	73.5
35.....	370	69	"	72.5	72.5
41.....	346	77	"	78.5	78.5
49.....	346	75.5	"	76.5	76.5
51.....	346	79.5	"	81.0	81.0
57.....	342	78.5	"	79.5	79.5
59.....	342	78.5	"	79.5	79.5
63.....	325	72	crumbling.	75.0	75.0
67.....	320	77.5	hard.	78.5	78 5

NOTE.—The temperature of the water in which bars were submerged was that of the water in St. Mary's River, about 50° Fahr.

The results, as observed from these tests are summarized in Table No. 14 and Fig. 1, from which deductions can be drawn by those interested in investigations of the theory of concrete.

Mr. Von Schon.

TABLE No. 14.—Tabulated Results.

Kind of Concrete.			Constant "C."	Absorption in per cent. of volume	Kind of Concrete.			Constant "C."	Absorption in per cent. of volume.
Cement.	Aggregate.	Mixture.			Cement.	Aggregate.	Mixture.		
E.....	S. S.	A	9.91	9.6	E.....	B. S.	A	18.16	4.8
E.....	S. S.	B	12.23	7.0	E.....	B. S.	B	18.44	3.2
E.....	S. S.	C	15.75	6.0	E.....	B. S.	C	20.93	3.2
E.....	S. S.	D	18.26	.....	E.....	B. S.	D	22.64	.....
E.....	S. S.	E	5.87	.....	E.....	B. S.	E	18.52	.....
R.....	S. S.	A	9.59	.....	R.....	B. S.	A	19.49	.....
R.....	S. S.	B	9.91	8.0	R.....	B. S.	B	20.80	4.8
R.....	S. S.	C	14.76	6.4	R.....	B. S.	C	21.98	3.2
R.....	S. S.	D	17.32	4.8	R.....	B. S.	D	22.05	3.2
R.....	S. S.	E	10.31	11.2	R.....	B. S.	E	18.09	.....
F.....	S. S.	A	4.12	.....	F.....	B. S.	A	8.12	.....
F.....	S. S.	B	6.57	.....	F.....	B. S.	B	9.78	.....
F.....	S. S.	C	8.96	.....	F.....	B. S.	C	9.30	.....
F.....	S. S.	D	6.79	.....	F.....	B. S.	D	8.03	.....
F.....	S. S.	E	7.29	.....	F.....	B. S.	E	6.84	.....
N.....	S. S.	A	0.00	.....	N.....	B. S.	A	2.89	.....
N.....	S. S.	B	2.23	.....	N.....	B. S.	B	3.05	.....
N.....	S. S.	C	1.87	.....	N.....	B. S.	C	3.72	9.6
N.....	S. S.	D	2.37	.....	N.....	B. S.	D	2.00	.....
N.....	S. S.	E	1.96	.....	N.....	B. S.	E	3.33	.....

Note.—Constant "C" $= \dfrac{\text{Span in feet} \times \text{Load in pounds.}}{\text{Width in inches} \times \text{Depth in inches}^2.}$

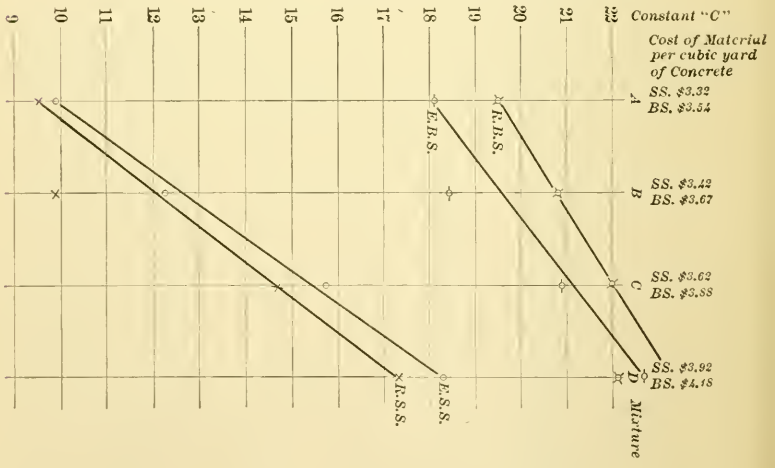

Fig. 1.

HISTORICAL SKETCH

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By Charles Warren Hunt, M. Am. Soc. C. E.

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At the Annual Meeting, January 19th, 1898, the following facts in regard to the subscription to this book were brought out:

Two thousand copies were printed; 300 were bound in full morocco, of which 216 have been sold at $10 per copy, the resulting net profit being $943.06. Seventeen hundred copies, which have been paid for, are still on hand, and the Board of Direction was requested to consider the propriety of offering to the membership these copies bound in a less expensive style and at a reduced price, the net proceeds to be applied to the building fund.

In compliance with this request it has been decided to bind as many copies as are necessary to supply the demand, in a handsome cloth binding and to supply them at $5 per copy.

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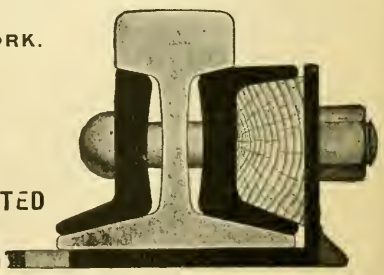

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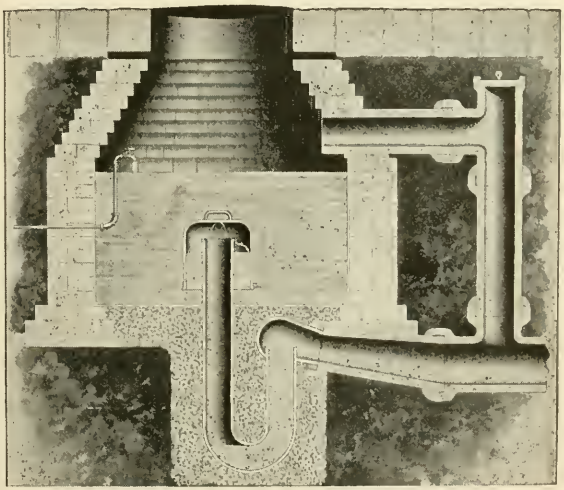

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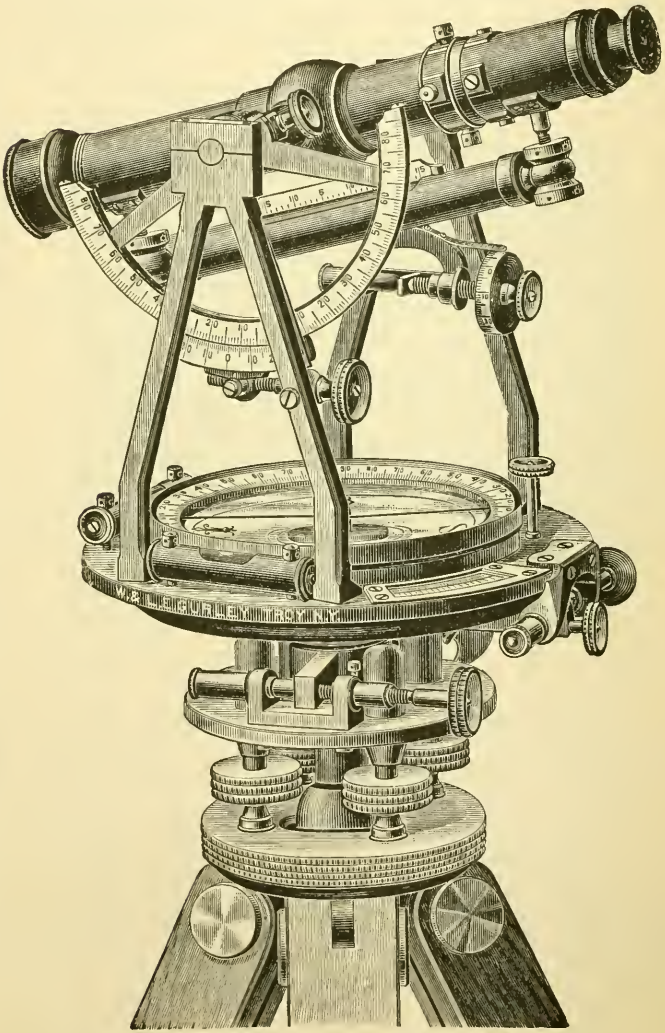

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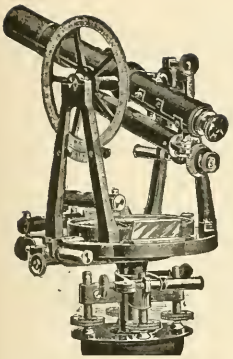

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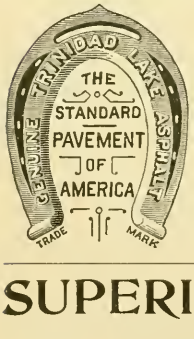

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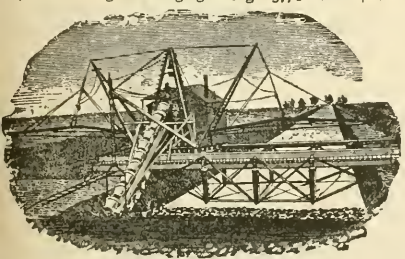

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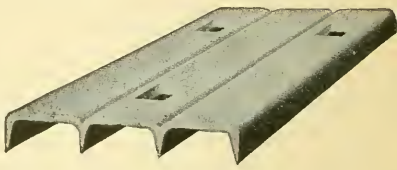

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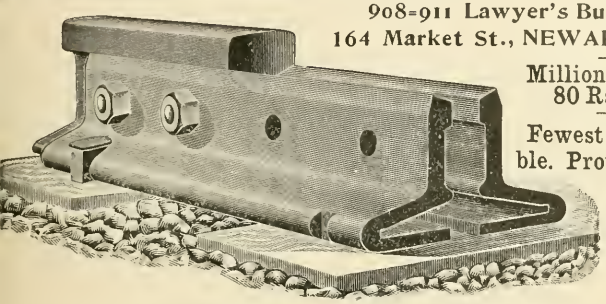

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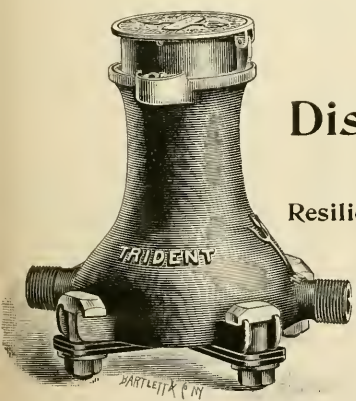

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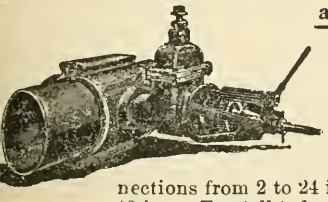

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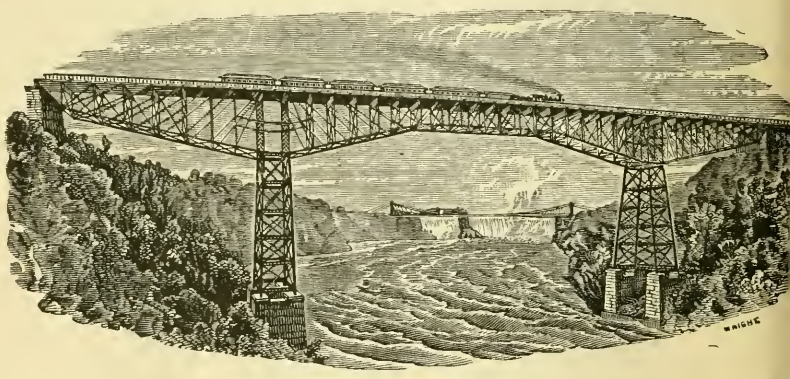

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AMERICAN SOCIETY

OF

CIVIL ENGINEERS

September, 1899

PROCEEDINGS = VOL. XXV—No. 7

Published at the House of the Society, 220 West Fifty-seventh Street, New York, the Fourth Wednesday of each Month, except June and July.

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ERRATUM.

Proceedings, Vol. XXV, Page 126 (August, 1899).

Insert after 11th line the following paragraph, which was omitted by mistake from the account of the Annual Convention :

In the evening Colonel Charles W. Raymond, Corps of Engrs., U. S. A., M. Am. Soc. C. E., addressed the Society on the subject of Breakwater Construction, with special reference to the Delaware Breakwater. The address, which was informal, was illustrated with lantern-slides, and proved of great interest to a large audience.

Vol. XXV. SEPTEMBER, 1899. No. 7.

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publications.

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American Society of Civil Engineers.

OFFICERS FOR 1899.

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FREDERIC P. STEARNS.	ROBERT MOORE.

Secretary, CHARLES WARREN HUNT.

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Special Committees.

ON UNIFORM STANDARD TIME:—Sandford Fleming, Charles Paine, Theodore N. Ely

ON ANALYSIS OF IRON AND STEEL:—Sub-Committee of the American Society of Civil Engineers (of the International Committee on Standards for the Analysis of Iron and Steel, of which Prof. J. W. Langley is Chairman)—Charles B. Dudley, William Metcalf, Thomas Rodd.

ON UNITS OF MEASUREMENT:—George M. Bond, William M. Black, R. E. McMath, Charles B. Dudley, Alexander C. Humphreys.

ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Noble, George S. Webster, O. M. Carter, W. B. W. Howe, Louis C. Sabin, H. W. York.

The House of the Society is open every day, except Sunday, from 9 to 22 o'clock.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER, - - - 533 Columbus.
CABLE ADDRESS, - - - - "Ceas, New York."

Vol. XXV. SEPTEMBER, 1899. No. 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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MINUTES OF MEETINGS.

OF THE SOCIETY.

September 6th, 1899.—The meeting was called to order at 20.40 o'clock, Mr. Elmer L. Corthell in the chair; Charles Warren Hunt, Secretary, and present, also, 68 members and 7 visitors.

The minutes of the meeting of June 7th, 1899, and of the Annual Convention were approved as printed in *Proceedings* for August, 1899.

A paper entitled "Pile-Driving Formulas: Their Construction and Factors of Safety," by Charles H. Haswell, M. Am. Soc. C. E., was presented by the author, and was discussed orally by Messrs. Foster Crowell, E. Sherman Gould, O. J. Marstrand, Horace J. Howe, John G. Tait and the author. The Secretary also presented a written discussion on the subject by Mr. L. J. Le Conte.

The second paper of the evening: "The Groined Arch as a Covering for Reservoirs and Sand Filters; Its Strength and Volume," by Leonard Metcalf, Assoc. M. Am. Soc. C. E., was presented by the author, and was discussed orally by Mr. Allen Hazen and the author. Written discussions on this paper by Messrs. W. R. Hutton and L. J. Le Conte were presented by the Secretary.

Ballots were canvassed and the following candidates declared elected:

As Members.

Reginald Pelham Bolton, New York City.
Edward Austin Bond, Albany, N. Y.
William Beverly Chase, Portland, Ore.
Arthur Truman Safford, Lowell, Mass.
John G Sullivan, Trail, B. C., Canada,

As Associate Members.

James Ferguson Burns, Hopkinsville, Ky.
Douglas Cornell, Buffalo, N. Y.
Joseph Hooker Cunningham, Portland, Ore.
Robert Delano Cushing, Minneapolis, Minn.
Fred Lavis, Chihuahua, Mexico.
Herman Julius Pfeifer, St. Louis, Mo.
Howard Crathorne Phillips, Topeka, Kans.
George Chandler Whipple, Brooklyn, N. Y.

The Secretary announced that at the meeting of the Board of Direction, September 5th, 1899, the ballot on the election of John Fritz, M. Am. Soc. C. E., to Honorary Membership was canvassed, and Mr. Fritz declared elected an Honorary Member of the Society.

Announcement was made that the following candidates were elected by the Board of Direction, September 5th, 1899.

As Juniors.

John Cornelious Sesser, Omaha, Neb.
George Rosengarten Sinnickson, Philadelphia, Pa.

The Secretary announced the following deaths:

Charles Carroll Gilman, elected Fellow May 11th, 1871; died July 31st, 1899.

Horace Harding, elected Member November 2d, 1892; died July 29th, 1899.

Albert Howell Porter, elected Junior July 4th, 1888; Associate Member July 1st, 1891; died August 9th, 1899.

Karl Spörck, elected Member March 2d, 1898; died June 23d, 1899.

Horatio Gouverneur Wright, elected Honorary Member March 3d, 1890; died July 2d, 1899.

The Secretary called attention to the invitation of the American Institute of Mining Engineers to members of this Society to attend the next meeting in San Francisco, as announced in *Proceedings* for August, 1899.

Adjourned.

September 20th, 1899.—The meeting was called to order at 20.40 o'clock, Director H. S. Haines in the chair; Charles Warren Hunt, Secretary, and present, also, 52 members and 7 visitors.

A paper by Henry S. Prichard, M. Am. Soc. C. E., entitled "Insufficient Provision for Counterstresses in Railroad Bridges," was presented by the author, and was discussed orally by Messrs. Henry B. Seaman, Oscar Lowinson and the author.

A written discussion on the subject by Mr. Henry S. Jacoby was presented by the Secretary.

The second paper of the evening by C. W. Hudson, Assoc. M. Am. Soc. C. E., entitled "Comparison of Weights of a Three-Hinged and a Two-Hinged Spandrel-Braced Parabolic Arch," was presented by the Secretary, who also presented a written discussion from Mr. Henry S. Jacoby.

The paper was discussed orally by Messrs. Henry S. Prichard and R. S. Buck.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

September 5th, 1899.—Vice-President Edward P. North in the chair; Charles Warren Hunt, Secretary, and present, also, Messrs. Clarke, Deyo, Hering, Manley, Owen, Ricketts, Schneider, Thomson and Whinery.

The Secretary announced the election of John Fritz, M. Am. Soc. C. E., by the unanimous vote of all members of the Board of Direction and all living Past-Presidents, as Honorary Member of the Society.

The appointment by the President of Messrs. Ricketts, Manley and Hunt, as a committee to report to the Board a recommendation as to the Time and Place for holding the next Annual Convention, was announced.

The following matters referred to the Board of Direction by the Annual Convention were considered:

The advisability of communicating with the Superintendent of the Twelfth Census in reference to the importance of collecting and publishing complete statistics of municipal works.

The propriety of providing for the expenses of the members of the Nominating Committee in attending the meetings of that Committee.

The matter of the revision of the Article of the Constitution relating to the nomination of officers.

The resolution that the Board be requested to take up and settle the matter of Standard Time.

Applications were considered and other routine business transacted.

Two candidates for Junior were elected.

Adjourned.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society is open every day, except Sunday, from 9 to 22 o'clock.

MEETINGS.

Wednesday, October 4th, 1899, at 20.30 o'clock, a regular business meeting will be held. Ballots for membership will be canvassed, and a paper by S. Bent Russell, M. Am. Soc. C. E., entitled "Impact Test of Structural Steel," will be presented. This paper is printed in the August number of *Proceedings*.

Wednesday, October 18th, 1899, at 20.30 o'clock, a regular meeting will be held, at which a paper by Lewis M. Haupt, M. Am. Soc. C. E., entitled "The Reaction Breakwater as Applied to the Improvement of Ocean Bars," will be presented. This paper is printed in this number of *Proceedings*.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(August 7th to September 13th, 1899.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible. Serial articles are not included.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Eng. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Eng., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Can.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Indicator*, Stevens Institute, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 30c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 35c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c
(14) *The Engineering Record*, New York City, 12c
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City, 35c.
(18) *Railway and Engineering Review*, Chicago, Ill.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England.
(22) *Iron and Coal Trades Review*, London, England.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gaslight Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Electrical Engineer*, New York City, 10c.
(28) *Industries and Iron*, London, England.
(29) *Journal*, Society of Arts, London, England.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des Ecôle Spéciales de Gand*, Brussels, Belgium.
(32) *Memoirs et Compt Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mecanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Transactions*, Am. Inst. Elec. Eng., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 75c.
(45) *Mines and Minerals*, Scranton, Pa. 20c.
(46) *Scientific American*, New York City, 10c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift des Vereines Deutscher Ingenieure*, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Duesseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereines*, Vienna, Austria.
(54) *Den Tekniske Forenings Tidsskrift*, Copenhagen, Denmark.
(55) *Ingeniören*, Copenhagen, Denmark.
(56) *Teknisk Tidskrift*, Stockholm, Sweden.
(57) *Teknisk Ugeblad*, Christiana, Norway.
(58) *Proceedings*, Eng. Soc. W. Pa. 410 Penn Ave., Pittsburg, Pa. 50c.
(59) *Transactions*, Mining Institute of Scotland, London and Newcastle-upon-Tyne.
(60) *Bridges and Framed Structures*, 358 Dearborn St., Chicago, Ill., 30c.
(61) *Proceedings*, Western Railway Club. 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.

LIST OF ARTICLES.

Bridge.

The Northfield, Mass., Bridge: Erection without Falsework. (14) September 2: (13) September 7.
Double Deck Bridge at Wells Street, Chicago. (13) August 24.
The Scherzer Rolling Lift Bridge. Fort Point Channel, Boston. (14) August 19: (15) September 8.
The Lewiston and Queenston Suspension Bridge. (14) August 26.
Plate Connections for Cylinder Piers. (13) August 31.
Failure of a Wooden Swing Bridge, Chicago. (18) August 26.
Removing a Bridge (Chicago). (40) August 18.
Tests of Frozen Concrete. (14) August 26.
Great Central Railway, Leicester to Rugby. (12) September 1.
Bridge Renewal in Ceylon. (12) September 1.
The New Thames Bridges. (12) September 1.
The Trans-Siberian Railroad. (46) August 26.
A Russian Drawbridge. (40) August 25.
Les Travaux d'Amelioration du Pont de Huy. E. Marote. (30) August, 1899.
Exécution d'un Pont Biais en Béton Armé d'après le Système Hennebique. E. Haerens. (30) August, 1899.

Electrical.

Magnetism and Molecular Rotation. Lord Kelvin. (27) August 26.
The Field of Experimental Research. Elihu Thomson. (27) September 9.
On the Determination of Current Strength in Three-Pointed Star Resistance Systems. A. E. Kennelly. (27) August 19.
Measurement of Power in Polyphase Systems. Aug. J. Bowie, Jr. (27) August 19.
The Protection of Low-Tension Wiring Against Dangerous High-Potential Currents. W. J. Plews. (24) August 7.
Some Methods of Measuring High Potentials with Low-Potential Instruments. Prof. Samuel J. Barnett. (27) August 26.
The Schattner Standard Electricity Meter. (47) July 29.
On the Predetermination of the Regulation in Alternating Current Transformers. A. E. Kennelly. (27) September 2.
Motor Starting Rheostats. Otto Mattick. (27) September 2.
A New Method of Determining Ratio of Transformation. J. R. Bibbins. (27) September 9.
The Sag and Tension of Line Wire. Cary T. Hutchinson. (13) August 31.
A 250 Horse-Power Electric Heating Plant. (27) September 2.
Portable Photometer as Used in the Standard Oil Company's Test. (13) September 7.
Wireless Telegraphy. James L. Adams, Jr. (27) August 19.
Concentric Wiring. Mervyn O'Gorman. (27) August 12.
The Storage Battery in Telephone Exchanges. (27) September 9.
Automatic Telephone Exchange at Augusta, Ga. Lee Campbell. (27) August 26.
An Electric Pantograph. Lieut. F. Jarvis Patten. (27) August 12.
Electric Engraving Machine. Lieut. F. Jarvis Patten. (27) August 26.
Arc Machine Regulation. H. Franklin Watts. (24) August 7.
Direct Alternating Generators. Alton D. Adams. (27) September 9.
Plymouth (Eng.) Lighting and Tramways. (26) August 11.
Barrow-in-Furness Electricity Works. (26) August 4.
West Ham Electricity Works. (26) August 25.
Gasoline Engine, 60-Kw., 220-Volt, Watertown, S. D., Central Station. Prof. F. W. Springer. (27) September 9.
The Economy Effected by the Use of the Electric Motor Around Steel Plants, Furnaces and Foundries. Andrew E. Maccoun. (27) September 9.
Electricity in the Mines at Cripple Creek. Thomas Tonge. (9) September, 1899.
Water-Power Electrical Plants in the United States. B. C. Washington, Jr. (3) September, 1899: (24) September 11.
Electric Haulage. J. W. Kirkland. (22) August 25.

Marine.

The Mechanical Theory of Steamship Propulsion. Robert Mansel. (12) August 18.
Experiments on Thrust-Block Friction. F. von Kodolitsch. (47) August 5.
The Manufacture of Propeller Shafts. D. B. Morison. (11) September 1.
Large Atlantic Cargo Steamers. G. B. Hunter. (11) August 18.
Untrammeled Shipbuilding and Marine Engineering Development. Joseph R. Oldham. (10) September, 1899.
The Development of German Shipbuilding. Rudolph Haack. (9) September, 1899.
Ice Breakers. H. F. Swan. (47) July 29.
The Equipment of Shipbuilding Sheds. (12) August 4.
The Twin-Screw Steamer *Bavarian* of the Allan Line (12) August 25.
Commodore Melville on Triple-Screw Warships. (12) August 4.
The Machinery of Warships Constructed at Keyham. Robert Mayston. (47) August 5.
Elswick Cruisers. Philip Watts. (11) August 4.
The Launch of a Battleship. H. R. Champness. (13) August 24.
Machinery of H. M. S. S. *Proserpine* and *Psyche*. Robert Mayston. (11) August 18.
On the Boiler Arrangements of Certain Recent Cruisers. F. T. Marshall. (11) August 11; (47) September 2.

Marine—(Continued).

The Boiler Trials of the *Sheldrake*. (12) August 11.
The Austro-Hungarian Cruiser *Zenta*. (12) August 25.
Drague Marine à Godets de 1 000 Chevaux pour le Port de Vladivostock. (33) August 19.
L'Avenir de la Construction Navale en Belgique. P. Moulin. (31) Tome xxii, deuxième livraison.

Mechanical.

Influence of the Length of Span in Transverse Tests of Cast Iron. Dr. R. Moldenke. (13) August 17. (52) August 15.
The American Foundrymen's Association Tests on Cast Iron as Related to Electrical Machinery. Dr. R. Moldenke. (27) August 26.
Mechanical Uses of the Science of Metallography. Albert Sauveur. (9) September, 1899.
Pig-Iron Fractures and Their Value in Foundry Practice. James Williard Miller. (47) August 19.
The Works and Operations of the Barrow-Haematite Steel Company. (22) August 4.
The Lorain Blast Furnace Plant. (22) July 28.
On the Use of Finely-Divided Iron Ore Obtained by Concentrating Processes. Professor J. Wiborgh. (12) August 18; (22) August 18; (13) September 7; (62) September 7; (16) September 9.
Pig-Iron Casting Apparatus. (12) August 4.
A New Casting Machine for Blast Furnaces. Richard Hanbury Wainford. (11) August 25; (47) August 26.
The Diffusion of Elements in Iron. John Oliver Arnold and Andrew McWilliam. (11) August 25.
Standards for Cast Iron. (11) August 25.
An Explanation of the Rapidity of the Bertrand-Thiel Process. Henry M. Howe. (16) September 2.
Importance of Fillets and Round Corners of Modern Machinery Castings. (19) August 12.
Iron in Ornament. Chester B. Albree. (10) September, 1899.
The Difference of Elasticity between Chilled and Ordinary Cast Iron. C. Bach. (48) July 22.
Some Results of Transverse and Tension Tests of Steel Plates. (15) September 8.
Notes on Rod Rolling and its Development. N. K. Turnbull. (22) August 25.
The Manufacture of Tin Plates—Past and Present. W. C. Cronemeyer. (22) August 25.
Drawing Press for Metal Sheets, 1 500 mm. diameter and 3 mm. thick. R. Kannegiefser. (48) August 5.
The Durability of Water-Tube Boilers. A. F. Yarrow. (11) August 4; (47) August 5.
Nickel Steel *versus* Mild Steel Boiler Tubes. A. F. Yarrow. (20) August 10.
The Phillips Water-Tube Boiler. (11) August 25.
Keen's Water-Tube Boiler. (11) August 18.
The Lynn Boiler. (52) August 24.
Mechanical Draft. Henry B. Prather. (1) July, 1899.
A New Method of Forced Draft. Nelson Foley. (47) July 29.
Uniform Rules for the Arrangement and Dimensions of Chimneys for Steam Plants. G. Lang. (48) July 29.
Economy in the Use of Superheated Steam. R. S. Hale. (9) September, 1899.
The Andrews & Martin Balanced Slide Valves. (11) August 11.
The Stresses in Steam Engines, with Special Reference to Electric Traction. (26) September 1.
The Thermal Efficiency of the Steam Engine. (12) September 1.
Connecting Rods for H. M. T. D. *Bullfinch* and *Dove*. (12) September 1.
Air Compressor Plant, General Post Office, Liverpool. (12) August 18.
Vertical Oil Engine. (47) August 12.
A "Fin-de-Siecle" Machine Shop. Fred Bathurst. (26) August 18.
Some Reasons for the Excellence of American Machinery. Henry B. Binsse. (10) September, 1899.
Modern Machine Tools. K. E. Smidt. (57) August 10.
New Power Plant for the Lancaster Mills. (14) September 2.
Elements of Design Favorable to Speed Regulation in Plants Driven by Water Power. A. V. Garratt. (24) August 7.
Transmission of Power by Means of Ropes and Belts. G. H. Thurston. (22) September 1.
The Transmission of Energy by a Partially Rarefied Atmosphere. Sir Frederick Bramwell. (12) August 18.
An Underground Pumping Engine at a Hungarian Colliery. Stefan Stener. (16) August 12.
Pneumatic Riveting. (15) September 1.
Fuel Gas. C. Francis Jenkins. (46) August 12.
Utilizing Charcoal Blast Furnace Gases. F. Zeyringer. (22) September 1.
Artificial Illumination for Ordinary Business and Domestic Purposes. Leon P. Lowe. (24) August 7.
The Automatic Ignition of Gas by Chemical Means. J. B. Grimwood. (24) August 21.
The Acetylene Exhibition and Congress at Budapest. (12) August 11.
Electrolysis. William A. Aldrich. (24) August 14.
The Automobile Situation. Hiram Percy Maxim. (10) September, 1899.

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Electric Automobiles. Elmer A. Sperry. (18) August 5.
A New Petroleum-Spirit Motor-Vehicle. (28) September 1.
The Kühlstein-Vollmer Motor-Tractor. (28) August 18.
New French Types of Automobiles. (27) September 2.
The New Mors Vertical Petroleum Spirit Motor. (28) August 4.
The Julien Friction Clutch for Motor Vehicles. (28) August 25.
Motor-Wagon Trials at Liverpool. (11) August 4; (12) August 4.
Commission's Report on the Test of Heavy Motor Vehicles, held at Versailles in October, 1898. (13) September 7.
Cycle Construction and Design. Archibald Sharp. (29) August 18.
L'État de la Vapeur à la Fin de l'Émission. Georges Duchesne. (37) July, 1899.
Des Semoirs en Lignes. Max Ringelmann. (37) July, 1899.
Les Progrès Actuels dans la Construction des Machines à Vapeur. Alfred Van der Stegen. (31) Tome xxii, deuxième livraison.
Installation de Chaudières avec Émulsèur à vapeur, à la Sucrerie Centrale de Cambraí. (33) August 5.
Fabrication Industrielle de l'Air Liquide, usine pour une Production de 700 litres par jour. Julien Lefèvre. (33) August 5.
Utilisation Directe des Gaz des Hauts Fourneaux dans les moteurs á Explosion. Aug. Dutreux. (33) August 26.
Le Moteur Diesel et les Moteurs Thermiques. Donat Banki. (33) August 19.
L'Éclairage par l'Alcool M. L. Denayrouze. (32) June, 1899.
Étude Supplémentaire sur la Production et sur l'emploir de Divers Gaz Combustibles. M. Lencauchez. (32) June, 1899.

Mining.

The Reduction of Metals not Easily Fusible at High Temperatures through Aluminum. M. Goldschmidt. (56) August 5.
A Furnace for Smelting Zincy Lead Ores. Ludwig Kloz. (16) September 2.
Coal Mining in Michigan. Charles Holmes. (45) September, 1899.
Notes on Silver Losses in Smelting. Malvern W. Iles. (16) September 9.
The Effect of Subsidence Due to Coal-Workings, upon Bridges and other Structures. (13) September 7.
Dust Explosions in Coal Mines. John Hayes. (24) August 14.
The Mining and Preparation of Kaolin. T. C. Hopkins. (16) August 26.
Mechanical Mine Haulage. (16) August 19; (45) September, 1899.
The Re-Lighting Marsant Mine Lamp. (16) August 19.
The Iron Mines of Cuba. (20) August 17.
Machine-Mining and Pick-Mining Compared. W. D. L. Hardie. (49) Vol. xx, Part 4.
Nouveau Procédé de Lavage des Sables Aurifères en Australie. G. Hamelin. (36) August 10.
Installation de Surface des Mines d'Or de la "French Rand Gold Mining Co." (Republique Sud-Africaine.) J. Dupont. (33) August 26.

Municipal.

Roadways Across Bogs. (14) August 19.
Repairs to Asphalt Pavements. (14) August 19.
Municipal Ownership of Tramways in Great Britain. Benjamin Taylor. (10) August, 1899.
Railway Asphalt Plant. (18) September 2.
Proposed Salt-Water Street-Sprinkling Plant at Oakland, Cal. (13) September 7.

Railroad.

Rails and Rail Joints. M. K. Bowen. (10) August, 1899.
The Breakage of Steel Rails. (13) August 31.
Hinged Switch and Cast-Steel Frog; Netherlands State Railways. (13) September 7.
Rapid Locomotive Erection in France. (11) September 1.
The Use of Nickel Steel in Locomotive Construction. (47) September 2.
American Locomotives in Great Britain. Charles H. Jones. (10) September, 1899.
American Locomotives on British Railways. Charles Rous-Marten. (9) September, 1899.
Cooke Ten-Wheel Freight Locomotive, Oregon Short Line. (18) September 2; (15) September 8.
New Passenger Locomotives for the New York Central. (40) August 18.
New Consolidation Engine of the Delaware, Lackawanna and Western Railroad. (15) August 25.
Heavy Ten-wheel Freight Locomotives for the Union Pacific. (15) August 25. (18) August 12; (25) September, 1899.
A Ten-Wheel Express Locomotive in England. (15) August 18.
Atlantic Type Locomotive for the Pennsylvania Railroad. (18) August 26; (46) August 26.
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Six-Coupled Locomotive, Cauca Railway, United States of Colombia. (18) August 12.
Consolidation Locomotives for the Delaware & Hudson Canal Company. (18) August 5.
Consolidation Locomotives, Santa Fé Pacific Railway. (25) September, 1899.
Ten-Wheel Passenger Locomotive, Denver & Rio Grande Railway. (25) September, 1899; (40) August 25.

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Two Improvements in Compound Locomotives. (25) September, 1899.
Six-Coupled Side-Tank Locomotive, Swedish State Railways. (25) September, 1899; (40) August 11.
Six-Coupled Narrow-Gauge Locomotive for Tasmania. (12) August 4.
A Ten-Wheel Express Locomotive in England. (15) August 18.
The Richmond Double-Ported Valve. (15) August 18.
A Locomotive Boiler with Corrugated Furnace. (13) August 24; (15) September 1; (25) September, 1899.
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Smokeless Firing with Bituminous Coal on Passenger Locomotives. (13) August 31.
The Electric Locomotive. George R. Mair. (10) August, 1899.
The Selection of Rolling Stock. C. F. Uebelacker. (10) August, 1899.
The Rolling Stock of the Great Central Railway. (11) August 25.
B. & O. Southwestern New 50-Foot Furniture Car and Freight Car Truck. (15) August 11.
A New Pile-Driver of the Chicago, Milwaukee & St. Paul. (15) August 11.
Special Steel Car for Transporting Heavy Loads; North Belgian Railway. (13) August 17.
Freight Cars for the Imperial Chinese Railways. (13) August 31.
The Development of the Street Car. John A. Brill. (10) August, 1899.
The Proposed Guayaquil and Quito Railway in Ecuador. (13) August 17.
The South Devon Atmospheric Railway. Sir Frederick Bramwell. (11) August 25.
North Bessemer Tunnel. F. E. House. (48) June, 1899.
The Ventilation of Tunnels. Francis Fox. (13) August 31; (14) September 2.
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Light Electric Railways. Dr. Louis Bell. (10) August, 1899.
Polyphase Alternating Currents for Electric Railways. Dugald C. Jackson, M. Am. Soc, C. E. (10) August, 1899.
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The City and South London Electric Railway. P. V. McMahon. (10) August, 1899.
Combined Operation of Trolley and Steam Systems on the Long Island Railroad. (27) September 2.
A Model American Electric Railway System. (11) August 18.
Storage Batteries and Electric Railways. Herbert Lloyd. (10) August, 1899.
Some Early Traction History. Thorburn Reid. (10) August, 1899.
Electrolysis from Railway Currents. Arthur V. Abbott. (10) August, 1899.
The Latest Developments in Electric Conduit Railways. F. S. Pearson. (10) August. 1899.
Electric Tramways in Great Britain. Robert W. Blackwell. (10) August, 1899.
Building an Electric Railway. Lemuel W. Serrell. (10) August, 1899.
Overhead Construction. J. G. White. (10) August, 1899.
The Development of the Modern Electric Railway Motor. Dr. Cary T. Hutchinson. (10) August, 1899.
The Conversion of Blackpool Tramways. (26) September 1.
The Stratford Trolley Car Disaster. (46) August 19.
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Traction Électro-Magnétique à Contracts Superficiels. (47) August 19.
Ventilation des Tunnels. Conditions d'Aeration. Volume d'air à Introduire dans le Tunnel et Travail Necessaire pour son Introduction. Raymond Godfernaux. (47) August 19.
De la Révision des Principes Appliqués par l'État-Belge en vue de la Protection des Points Dangereux. H. Van den Bogaerde. (31) Tome xxii, deuxième livraison.
Étude sur les Vibrations des Bielles d'Accuplement. F. Keelhoff. (31) Tome xxii, deuxième livraison.
Quelques Résultats d'Essai des Principaux Injecteurs pour Locomotives employès a l'État-Belge. Ernest Grootaert. (31) Tome xxii, deuxième livraison,
Le Cintrage des Bois aux Ateliers de la Compagnie de l'Est, à Romilly-sur-Seine. (34) August, 1899.
Tramway Électrique à Contacts Superficiels, Système Diatto. Pièrre Guédon. (34) August, 1899.
Les Freins Continus. R. Seguela. (37) July, 1899.

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Nouveau Système d'Éclairage Électrique des Voitures au moyen d'une Dynamo Actionnée par l'un des Essieux. M. Auvert. (38) August, 1899.
Lorry de Secours en cas de Déraillement des Voitures de Tramways Mécaniques. MM. Mauclère et Leroux. (38) August, 1899.
Notes sur les Corrosions, Cassures et Fissures de Toute Nature, Observées dans les Chaudières de Locomotives de la Campagnie des Chemins de Fer de l'Est. M. Desgeans. (38) August, 1899.

Sanitary.

The Proper Size for Septic Sewage Tanks. (13) August 17.
Septic Tank and Sewage Filter Beds for the Essex Co., Lunatic Asylum, Verona, N. J. (13) September 7.
A Small Septic and Nitrifraction Sewage Treatment Plant near Chicago. Alvord and Shields. (13) September 7.
Improved Methods for the Purification of Sewage and Water as Shown in the Operation of the Municipal Plant at Reading, Pa. John Jerome Deery. (3) September, 1899.
The Present Condition of Sewage Disposal at Paris. (13) August 17.
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The Influence of Sewage on Cement. (14) August 19.
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Refuse Destructor and the Results Obtained from a Six Months' Working of the Refuse Destructor at Torquay. Henry A. Garrett. (11) August 18; (47) September 2.
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Refuse Furnaces. (14) August 26.
Refuse Destructor Trials, Torquay, England. (14) September 2.
The Renovation of Bombay. (12) August 25.

Structural.

Foundations for Heavy Buildings. William H. Burr, M. Am. Soc. C. E. (10) September, 1899.
Reactions for a Beam Supported at Four Points. R. Birkeland (57) August 17.
Calculation of Cast-Iron Columns. J. B. Nau. (13) August 31.
The Relation Between the Structure of Steel and Its Thermal and Mechanical Treatment. Albert Sauveur. (13) August 24.
The Work of the International Association for Testing Materials. Mansfield Merriman, M. Am. Soc. C. E. (13) August 17; (15) August 18; (14) August 19.
The L. L. Sterns Stable, New York. (14) August 19.
The Mechanical Plant of the Exchange Court Building, New York. (14) September 9.
The Patent Office New Building. (12) August 18.
The Project to Warm Buildings from a Central Station in Providence. (14) August 19.
A Ventilated Chimney Tower. (14) August 19.
A Hospital Hot-Water Heating Plant. (14) August 26.
Upward and Downward Ventilation. (14) August 26.
Supports Horizontaux Armaturés, en Fer et en Ciment. Henry Pieq. (35) August, 1899.

Topographical.

A Revolving Camera for Surveying Purposes. (13) August 24.
Field Notes of a Civil Engineer—Do They Belong To His Client or To Himself? J. Van der Hoek. (1) July, 1899.
A New Optical Distance Meter, "Quadruple Micrometer." A. Tichy. (53) July 21.

Water Supply.

Twin Gates for the Fay Lake Reservoir, Arizona. (13) September 7.
Covered Reservoirs and their Design. Freeman C. Coffin, M. Am. Soc. C. E. (1) July, 1899.
Covered Reservoirs at Pasadena, Cal. T. D. Allin. (13) August 17.
The Assouan Dam. (12) August 11.
East Canyon Creek Dam, Utah. M. S. Parker, M. Am. Soc. C. E. (14) September 2.
The Water-Works of Cambridge, Mass. (14) August 26.
Auxiliary Water Supplies. (14) September 2.
Three Engineering Reports on the Ramapo Project. (14) September 2.
The London Water Supply—Its Future. (12) August 25.
The Water Supply of the City of Glasgow. Benjamin Taylor. (10) September, 1899.
New Tunnels, Intake Crib and Pumping Stations, Chicago, Ill. (13) August 31.
Additional Water Supply Works for Dresden, Germany. (13) September 7.
The Rice High-Pressure Pump. (20) August 31.
The Development of American Water Powers. (14) August 19.
The Mount Whitney Water Power Electric Plant and Light Regulating Device. (13) September 7.
Some Irrigation Appliances in the San Bernardino Valley, Cal. (13) September 7.

Waterways.

Methods used in Placing the Skeleton Steel Beacon Coast Signals. (13) September 7.
Lighthouse Illuminants. Lawrence Irwell. (19) September 9.
The Improvement of the Big Sandy River. (14) September 9.
The Dortmund-Ems Canal, and the Hydraulic Lock Lift at Henrichenburg. B. Gerdau. (48) August 12.
Recent Hydrographical Surveys. (12) August 11.
Moyens Mécaniques de Navigation Intérieure. M. Galliot. (37) July, 1899.

NEW BOOKS OF THE MONTH.

Unless otherwise specified, books in this list have been donated to the Library by the Publisher.

LOCKS AND LOCK-GATES

For Ship Canals. By Henry Goldmark, M. Am. Soc. C. E. Paper, 9½ x 6½ ins., 19 pp. (Donated by the Author.)

This is a reprint from the *Transactions* of the Association of Civil Engineers of Cornell University, and treats of the history, construction and operation of locks and lock-gates for ship canals. Two page plates of illustrations of the St. Mary's Falls Canal accompany the text.

MECHANICAL DRAWING.

A Practical Course in Mechanical Drawing. By William Fox and Charles W. Thomas. Cloth, 8 x 5½ ins., 98 pp., illus. New York, D. Van Nostrand Company. $1.25.

The object of this work is to provide a simple, practical course of progressive lessons in mechanical drawing. Instruction is given in connection with a special concrete exercise, instead of being presented in an abstract and general manner, as it is the belief of the authors that the subject is best taught with the use of models, instead of sketches and pictorial illustrations. The exercises are elementary and progressive in character, and the course is designed for high schools, schools for apprentices and young mechanics. The studies are intended to fit the pupil to make sketches from machinery and to enable him to read and understand an ordinary mechanical drawing.

POTABLE WATER

And Methods of Detecting Impurities. By M. N. Baker. Boards, 6 x 4 ins., 97 pp. New York, D. Van Nostrand Company. 50 cents.

This book, which is issued as No. 61 of the Van Nostrand Science Series, replaces an earlier issue, bearing the same number and practically the same title, by another author. It has been written without regard to the contents or methods of the preceding work, now 20 years old, and it aims to present, for the benefit of water commissioners and other officials, physicians, public-spirited citizens, and students in technical and medical schools, a clear statement of the essential qualities of potable water, how it may be obtained, and the significance of chemical, bacterial and microscopical tests of its quality, both in themselves and relatively. The relation between water and disease, and typhoid fever records as an index of the purity or impurity of public water supplies are also pointed out.

PRECISE SURVEYING AND GEODESY.

Elements of Precise Surveying and Geodesy. By Mansfield Merriman, M. Am. Soc. C. E. Cloth, 9 x 6 ins., 261 pp., illus. New York, John Wiley & Sons. $2.50.

This book sets forth the history of geodesy and the elements of geodetic theory and practice. The chapter headings are: The Method of Least Squares, Precise Plane Triangulation, Base Lines, Leveling, Astronomical Work, Spherical Geodesy, Spheroidal Geodesy, Geodetic Co-ordinates and Projections, Geodetic Triangulation, The Figure of the Earth, Tables. The tables include mean celestial refraction, lengths of arcs of the meridian, lengths of arcs of parallels, logarithms for geodetic computations, logarithms for the *L M Z* problem and constants and their logarithms. There is an index of 5 pages, and the pages carry on their inner margin folios relating to the chapter and article.

THE DESIGN AND CONSTRUCTION OF DAMS,

Including Masonry, Earth, Rock-fill and Timber Structures; also the Principal Types of Movable Dams. By Edward Wegmann, M. Am. Soc. C. E. Cloth, 12 x 9½ ins. 250 + xii pp., illus. New York, John Wiley & Sons. $5.00.

This is the fourth edition of Mr. Wegmann's book originally published as a treatise on the design and construction of masonry dams, and the work has now been enlarged to include also earth, rock-fill and timber structures and the principal types of movable dams. The original book, forming Part I of the present work, has been added to by the insertion of detailed description, drawings and views of some of the high masonry dams recently built to form storage reservoirs for the water supply of the City of New York, and by the publication in the Appendix of the specifications for the new Croton dam, the highest structure of its kind. Part II treats of the fixed dams mentioned, other than masonry, and Part III is devoted to movable dams. The work is illustrated by 75 figures in the text and 97 plates, including folders and half-tones. There are 24 Tables and a bibliography, divided in accordance with the general scheme of the book, in the Appendix.

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DEATH.

PORTER, ALBERT HOWELL .....Elected Junior July 4th, 1888; Associate Member, July 1st, 1891; died August 8th, 1899.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

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PAPERS AND DISCUSSIONS.

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THE REACTION BREAKWATER AS APPLIED TO THE IMPROVEMENT OF OCEAN BARS.

By LEWIS M. HAUPT, M. Am. Soc. C. E.

TO BE PRESENTED OCTOBER 18TH, 1899.

RESULTS TO DATE.

In a paper published by the Society in the *Transactions** for December, 1898, attention is called to the single curved breakwater at Aransas Pass, Texas, designed and located by the writer for the purpose of creating at that inlet a navigable channel at least 20 ft. in depth.

In the discussion which followed that paper it appeared that the incompleted breakwater was doing effective work; but as no survey had been made for some time, and the Harbor Company had suspended all work since May, 1897, it was impossible to state the results with any degree of accuracy. The latest examination available at that date was that made under the direction of a Board of Engineers for the purpose of appraising the value of the work done, with a view of turning it over to the Government to continue. This board reported on November 22d, 1897, that:

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in *Transactions*.

* "Origin of the Gulf Stream and Circulation of the Waters in the Gulf of Mexico, with Special Reference to the Effect on Jetty Construction," by N. B. Sweitzer, Jr., Jun. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xl, p. 86.

"The depth has not been materially, if at all, increased." * * * "At the time of examination of the Board, in July, 1897, it was 8.8 ft., and in November, 1897, 8.5 ft. The depths and contour lines are shown on appended charts."

(These charts, however, show a least depth at one crossing on the crest of the bar of 9¼ ft., November 2d to 5th.) The Board also expressed the opinion that "there does not seem any probability that the jetty as now constructed will of itself secure and maintain any considerable increase of depth in a navigable channel of proper width."

In view of this expectation and the fact that up to the end of February, 1899, a period of 15 months, the unaided currents have increased the depth at this particular part of the bar from "8½ ft.," as reported, to over 22 ft., and that, notwithstanding the existence of a large part of the old Government jetty across the channel, the question may pertinently be asked, whence these phenomenal results in so short a time?

Reaction vs. Concentration.

The answer is plain, since the curved form of the breakwater (not "jetty"), is such as to develop the potential energy of the ebb currents, rendering them kinetic, and applying them locally on the crest of the bar where they are needed for scour. At the same time, the breakwater serves to arrest the littoral drift and defend the channel from its encroachments. In short, it aims to utilize the well-known centrifugal force by the reaction developed by the resistance caused by a continuous change of direction so often manifested in the concave bends of streams, instead of the principle of concentration which has been relied upon so largely in the system of twin or convergent jetties, with such meager results.

This may be well illustrated, perhaps, by the case of a projectile flying through the air with a large amount of stored energy, which is developed suddenly only when meeting with a point of resistance. It has also been observed that currents having a velocity sufficient to scour the material over which they are flowing do not disturb it, but if a resisting log, rock or other obstruction is introduced, the obstruction is undermined and often buried and covered up by the reaction which it develops in the currents.

In Ex. Doc. 78, 48th Congress, it is stated that "The mean ordinary

velocity at the Narrows (New York Bay), is, during the ebb tide, about 2 ft. per second, and from this a depth of 100 ft. results." This does not follow, as there are numerous sandy bars having a much greater mean velocity over them which lie at depths less than 10 ft. from the surface. Manifestly, it is not velocity, therefore, that constitutes the working force to produce scour, but reaction, just as when impounded waters are dammed back prior to escaping through a breach, when the increased head and contraction react upon the bottom, affecting the entire volume and producing deep eddies. In the case of a single obstructing pier head, the effects will often extend to depths of 50 ft. or more, depending on the intensity and direction of the currents, scouring out the material and depositing it in the lee of the pier. In the case of the Narrows, above cited, it is not the ebb which scours, but the flood current, the resultant of which is near the bottom and which runs for eleven hours out of twelve up stream or into the Upper Bay, indicating a huge vertical eddy.*

The principle of concentration or trailing by convergent or parallel jetties, on the other hand, has been applied frequently by maritime engineers, but generally without producing the desired results unless aided by dredging. The most successful instances of this method are those at the mouths of large rivers which debouch into nearly tideless seas, as at the Sulina mouth of the Danube, built by Sir Charles Hartley; the South Pass, by the late James B. Eads, M. Am. Soc. C. E., and the Tampico jetties, by E. L. Corthell, M. Am. Soc. C. E.; but for purely tidal entrances the effect of the two jetties is to invert the natural trumpet-shaped opening and to diminish the area of the gorge, which is transferred to the crest of the bar, thus reducing the tidal volumes, preventing the complete filling of the interior compartment and requiring the same volume to move in both directions over the same path. The effect, therefore, is to minimize the movement and reduce the resultant to zero. Probably the most successful tidal harbor improved by convergent jetties is that of Dublin, where the improvement is largely due to a gap of 600 ft. left in the "Great North Wall," or jetty, near its shore end, to admit the flood tide which passes over the North Bull Sands, and which flows out chiefly through the jetties because of the form of the basin and the great range of the tides, which is there about 13 ft.

* Ms. Report of Professor Henry Mitchell, U. S. C. & G. Survey Office, or "Harbor Studies" by the author. Library, Am. Soc. C. E.

The following extracts from a paper read May 20th, 1879, by John Purser Griffith, Assoc. M. Inst. C. E., on "The Improvement of the Bar of Dublin Harbor by Artificial Scour," will serve to illustrate this point :

"Mr. Giles recommended that an opening should be left at the shore end of the proposed wall 600 ft. wide, to allow of a free passage for the tidal waters north of the Green or Bull Island, as the sand island on the North Bull was called. This opening had been proposed by Captain Corneille in 1802; but at that time the Directors General of Inland Navigation feared that the flood tide flowing through this opening would carry sand from the North Bull into the harbor, and that the injury thus done to the port might more than counterbalance any advantage to be gained by the opening.

"In 1835 Sir William Cubitt reported upon the state of the harbor. Referring to the improvement of the bar, he says: 'The great increase of depth and improved channel over the bar I attribute entirely to the erection of the Great North Wall, a measure founded upon sound principles, and carried into effect in a manner well calculated to effect the desired purposes—viz., that of checking the influx of sand upon the flood tide from the North Bull into the harbor, and giving an increased impetus at the ebb tide by means of narrowing the stream, and confining it to a direction suitable for keeping open the best channel, the effect of which is already shown by an increased depth of 5 ft. over the east bar since the erection of the Great North Wall.'

"Captain Washington, in his report, written in 1845, on the harbor of Dublin, as one of the Tidal Harbors Commissioners, referred to the Great North Wall as follows: 'The propriety of this measure, which involved so heavy an expenditure, has been a subject much controverted amongst persons connected with the port; but it is believed that there is now but one opinion as to its beneficial effects, and that the plan evinced both good judgment and skill.'

"There are few, if any, similar cases of a bar and entrance channel to any harbor being increased in depth like that of Dublin, viz., about 7 ft. in thirty years, and I think great credit is due to those who designed, as well as those who executed works which have achieved so important a result.

"The improvement of Dublin Harbor entrance ranks second to none. There is no other example, so far as the author is aware, of the construction of an artificial estuary for scouring purposes which has proved so successful.

"In the discussion Mr. Abernethy, Vice-President, said he thought attention should be directed to the construction of works which would tend to increase the tidal volume, and at the same time to prolong the action of the outgoing currents from the period of half ebb towards low water.

"Mr. Bergeron stated it was very difficult, almost impossible, to dredge sand bars in an open sea.

"Mr. Vernon-Harcourt said that to diminish the tidal capacity within a harbor was the worst thing that could possibly be done.

"Mr. Griffith said: 'Prior to the construction of the Great North Wall the ebb of a spring tide attained a velocity of $1\frac{1}{4}$ miles per hour across the bar, while at present it reached nearly $3\frac{1}{4}$ miles, showing an increase of about 2 miles per hour. He did not believe that very high velocities were efficient. Several instances might be named in which high velocities were attained, and yet the scour was a failure.

"Mr. Stoney thought that though there might be some doubt as to the authorship of the Great North Wall, there could be none as to its complete success."

From this quotation it appears that some of the best authorities in the world recognize an increase in depth of 7 ft. in thirty years as being beyond precedent, yet at Aransas Pass, with an incompleted reaction breakwater, having large gaps at both extremities, a feeble tide (only 14 ins.), and a serious submerged obstruction almost completely blocking the channel, the depths have progressively increased from about 6 ft. to an average of over 18 ft. in a few years, while in some places there are depths of 23 ft.; so that instead of a deepening at the rate of 1 ft. in over four years, the figures are reversed and show over 4 ft. in one year, a rate which is about twenty times greater than the best heretofore recorded.

These results are the more remarkable as they have been obtained without the aid of dredging, and on a bar which is composed of hard, fine sand, so compact that a boat hook or the blade of an oar does not penetrate it; so solid is it, that a Board of Engineers, in 1889, said of this bar that "The necessary results could not be obtained without extensive and almost constant dredging, which would involve incalculable expense."

As early as 1871, the project of obtaining a navigable channel over the Aransas Bar was pronounced impracticable because of its cost. The report of the United States Engineers of 1871 stated that "The cost of building a jetty, from Mustang or St. Joseph's Island towards the bar, which would be able to resist the action of the storms upon the quicksand foundation, must be an insuperable objection to any such experiment;" yet it was finally considered expedient, and it was further estimated in 1879 that, to obtain a 12-ft. channel would cost $759 185. (The ruling depths are 8 to $8\frac{1}{2}$ ft.) Work was commenced

on this project in 1880 and, after an expenditure of $550 416, up to June, 1890, it was discontinued, with a resulting depth of 7½ ft. In 1887 the project was modified "to secure and maintain a channel depth of not less than 20 ft.," and the same report stated that "The original estimated cost of this work, as here revised, is $2 052 543.72."

Cost of the Reaction Breakwater.

As the Government abandoned the work at Aransas and concentrated its appropriations on Galveston, a private company, known as the Aransas Pass Harbor Company, was incorporated, March 22d, 1890, and soon thereafter secured a large land bonus and proceeded to construct another jetty which, like its predecessor, was erroneously located on the southerly side of the Pass. It proved to be a failure, as was predicted, but seemed to be a necessary part of the experience of the Company.

At a later date the Company was reorganized, and it was then decided to give the reaction breakwater a trial by building only half of it, for which half depths of 15 ft. were predicted. Work was commenced in August, 1895, and in a few months the bar deepened to 13 ft.; but, unfortunately, the remains of the original Government jetty were then uncovered and found to be still in place, lying across the proposed channel, although previously officially reported to have "disappeared." The building of the breakwater to high water intersected this old work and formed a pocket which arrested the sand and prevented scour, while the Harbor Company failed to remove the obstructing jetty, as advised to do, until the winter of 1896–97, when a contract was made with Col. C. P. Goodyear to complete the work, he to advance all the funds and assume all risks.

He exploded some 23 350 lbs. of dynamite on the old jetty and bar, causing a breach of about 500 ft. in length near the breakwater, but being unable to secure the capital in time to continue the work, he was reluctantly obliged to surrender his contract in May, 1897. Since that time no work has been done. About ¼ mile of the breakwater near the center was built to full dimensions; the shore flank was extended to its minimum limits, but of a contracted section, with large gaps through which the ebb currents escaped seaward and the sand entered the channel during flood tide, and the mat was laid at the outer end to a distance of 1 000 ft. All this work was done by Charles

Clark & Company, of Galveston, and notwithstanding the storms, which have been violent, and the alleged quicksand, there is no appreciable loss or shrinkage in the sections of the structure, which has stood more than three years, in a very exposed position, doing excellent service.

The total length of the structure as designed is 6 200 ft., and about $250 000 has been expended on it.

The quantities were:

Rip-rap (core)	47 246 cu. yds.
Cap stones	9 183 "
Mattress work	17 000 "
	73 429 "

Say 75 000 cu. yds. at $3.33 per yard, and the cost per foot of depth gained, to date, has been approximately $25 000. It would, doubtless, have been much less but for the old jetty, which is still a serious obstruction and menace in the navigable channel.

As the total length of this work is less than half that required by the parallel-jetty system, it is evident that its cost will be much less while its efficiency is much greater, as it creates and maintains its own channel.

An official report of a thorough survey made in January and February, 1899, by the United States Coast and Geodetic Survey states, *inter alia:*

That during the winter the weather was so bad that it was only practicable to get on the bar for survey purposes about one day out of each week, and that "very heavy gales prevailed with terrific force with the wind from the north-north-east."

"In 1895 the Aransas Pass Harbor Company constructed a jetty in the shape of the letter S on the north side of the entrance, which is still in existence, and ever since its construction there has been a marked increase in the depth of water on the bar. The present channel crosses the Mansfield jetty, portions of which are still in existence. An attempt was made to remove this by explosion of dynamite, with the result that the rocks scattered over considerable area, and without doubt they prevent the current from scouring the bottom to its full capacity. * * * It is my opinion that by the completion of the present jetty and the clearing away of the rock in the entrance that a channel of at least 20 ft. in depth would soon be secured."

This report, taken as a whole, confirms in every particular the pre-

dictions of the inventor as to the results to be expected and the features embodied in a breakwater of this peculiar form and position.

The conditions which this structure was designed to fulfill were:

1. That it should arrest the littoral drift of sand and defend the channel from its encroachments—hence it must be placed to "windward" of the proposed channel.

2. That it should freely admit the full tidal prism during the entire period of flood tide, that there might be no reduction of volume at ebb; consequently it was detached from shore, there being a gap of nearly 1 800 ft. left open at the inner end.

3. That it should catch and control the greater part of the discharge at ebb and confine it locally to a narrow path across the bar, thus changing the conditions of equilibrium of the tidal movements largely in favor of the ebb.

4. It should be a compound and reverse curve, concave to the channel, and have such radii as to produce the best dimensions of waterway consistent with the volume of discharge, so as to develop a continuous reaction entirely across the bar and at the same time afford a good navigable channel.

5. It should also be so adjusted, with reference to the axis of the discharge, as to receive the same tangentially and train it across the bar without creating abrupt points of incidence, causing inequality of depths, and, at the same time, not producing so much compression as to cause undermining and ultimate destruction or large expense for maintenance.

6. It should also control a sufficient volume to prevent any encroachment of drift from the opposite side of the channel during the season of change in the direction of angular wave movements, and thus construct its counterpart of sand to aid in controlling the currents, without cost.

7. It should not promote the growth of the bar seaward by the deposit of the material removed from the channel on the outer slope of the bar.

8. It should rise above high water to afford an aid to navigation by breaking the waves and indicating the position of the channel.

9. It should be so located as to develop the channel across that part of the entrance where the bar-building forces are weakest, thus giving quicker and more permanent results.

That this single, detached, reaction breakwater fully satisfies the foregoing requirements, even under adverse circumstances and in an incomplete state, is demonstrated sufficiently by the work done at Aransas Pass, as shown by the progressive deepening and enlargement of the channel since the breach was made in the old Government jetty by Colonel Goodyear in 1897. That it is a stable, permanent and economical structure is also shown by the condition of the work after tests by severe storms and by years of exposure. That it has a remarkable power of producing erosion is shown by the fact that in February, 1895, the distance across the bar between the 15-ft. contours on the inside and outside was 3 650 ft., or nearly $\frac{3}{4}$ of a mile, which has been cut through except a few hundred feet, giving a navigable channel at mean low water of over 14 ft., and without material advance of the bar seaward.

It is not doubted by competent authorities that if the breakwater had been completed in the ten months, as originally proposed, and the old jetty had been promptly removed when discovered *in situ*, the predicted 20 ft. would have been secured ere now, and without aid from dredging, while the channel would have been self-maintaining.

In short, this simple structure marks a distinct advance in the resources of maritime engineers in the improvement of ocean bars, with great economy in cost of construction and maintenance, and large saving of time. It is the result of about 25 years of research and over 11 years of effort to secure a practical demonstration, and it has now reached a point sufficient to prove its great utility.

It is but just to add that these unprecedented results were probably unknown to the Board of Engineers which stated in 1897 "that the value to the Government of the works of the Aransas Pass Harbor Company for the improvement of Aransas Pass, Tex., is nothing," and that the same Board in reporting on the same subject December 17th, 1898, therefore urged the removal of the outer 1 000 ft. of foundation; the construction of a sill 40 ft. wide and 3 ft. deep, which would have obstructed the tides by closing the opening at the shore end; the construction of a parallel straight jetty on the southerly side of the channel and other works, supplemented by the dredging of 750 000 cu. yds. at an estimated cost of $1 525 000. This estimate was met by a proposition from a responsible firm of contractors to guarantee the 20-ft. channel within 2 years at a cost of less than half of the above

sum, with the result that Congress concluded to appropriate $60 000 to remove the obstructing portion of the old Government jetty, and permitted the incomplete breakwater to remain unmolested, that it might continue to deepen the channel by natural agencies and so further demonstrate its utilities. At that date the reported depth was 13 ft. 4 ins., while since then it has increased, within a few months, to about 18 ft. at the same point, thus confirming the wisdom of this action. With the gaps closed and the structure completed, there should result, in a short time, the full depth of 20 ft., as predicted, and at a cost considerably less than half of that of the parallel jetties, as originally proposed. The manifest economies of this method should suffice to commend it for more general use in the improvement of bars on alluvial coasts where there is a resultant littoral drift.

The writer desires to acknowledge his indebtedness to Major H. C. Ripley, M. Am. Soc. C. E., and George Y. Wisner, M. Am. Soc. C. E., consulting engineers, for assistance in preparing the specifications and for their unshaken confidence in the results to be expected from this plan from its inception, as well as to Colonel C. P. Goodyear for his successful effort to breach the old jetty, and thus enable the partially controlled currents to reach the crest of the bar.

The work was done under the efficient supervision of Major William Dunbar Jenkins, M. Am. Soc. C. E., resident engineer for the Harbor Company, and his assistants, Messrs. Pitts and Collins.

APPENDIX.

THE THEORY OF THE REACTION BREAKWATER.

It was the purpose of the writer to submit only a brief statement of physical conditions and results secured under very adverse circumstances, but, in deference to the opinions of the Committee on Publications, a digest of the researches which led to the development of this form of breakwater is added to supplement the subject.

The numerous and conflicting conclusions, reported after careful instrumental surveys made to determine the direction and intensity of the forces operating upon ocean bars, led the writer to give them but little weight, and to conclude that the best guide in such cases was to be found in a correct interpretation of the resultant hydrographic features as revealed by a study of comparative charts. This method is best applied to alluvial coasts where the material, being more or less mobile or plastic, is moulded by the ever-varying forces of winds, waves, tides and currents, the resultant effect of which is manifested by the form, size and location of the adjacent and submerged shoals, dunes and other features; in short, it is a deduction of cause from effect and is characterized by certain typical forms and directions of movement, so that a comprehensive as well as detailed study of the records covering an extended period of time are important factors in solving problems of this class.

The writer, therefore, gave considerable time to the investigation of the literature and charts pertaining to harbor inlets, both at home and abroad, making personal visits to the more important works, and was surprised to find how small a percentage were designed with a view to utilize the natural forces for deepening and maintenance. He prepared several papers discussing and classifying these problems, which have appeared in various scientific periodicals.*

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* "Harbor Studies." *Proceedings*, Engineers' Club of Philadelphia, Jan. 16th, 1886.

"The New York Entrance." *Proceedings*, Engineers' Club of Philadelphia, Feb. 6th, 1886.

"The Delaware Breakwater." *Proceedings*, Engineers' Club of Philadelphia, Feb 20th, 1886.

"Problem of the New York Entrance." *Proceedings*, Am. Assoc. for Advancement of Science, Aug. 20th, 1886.

"Improvement of Tidal Rivers." *Journal* of the Franklin Inst., Sept., 1887.

"Improvement of the Port of Philadelphia." *Journal* of the Franklin Inst., Oct., 1887.

"The Physical Phenomena of Harbor Entrances: Their Causes and Remedies." *Proceedings*, Amer. Phil Soc., 1888. (This paper received the award of the Magellanic Premium, Dec. 16th, 1887.)

"Discussion of the Dynamic Action of the Ocean in Building Bars, etc., being a reply to Report of the Board of Engineers, U. S. E." *Proceedings*, Amer. Phil. Soc , March, 1889.

(*This list continued in the foot-note to page 517.*)

The result of these investigations led to the following conclusions:

(1) That in many places there were cyclic movements of the bars and channels produced by the resultant of the external littoral forces which were the bar-building agents, and which predominated over the internal or ebb currents which were the bar-destroying elements.

(2) That maritime engineers, in their efforts to improve purely tidal entrances, had greatly reduced the tidal energy by constructing two parallel or convergent jetties from the shore line to the bar, thus obstructing the free ingress of the tide upon which they were dependent for both volume and velocity at ebb, and that such construction confined the movements at all stages of both ebb and flood to the same path across the bar, thus producing no changes of equilibrium in favor of the ebb and requiring constant expense for bar removal and maintenance between jetties by dredging.

(3) That in the construction of jetties, where there is a resultant littoral drift, the wrong one is sometimes built first, thus arresting the drift and causing the bar to advance seaward (in one case nearly 3 miles), thus greatly increasing the cost.

(4) That no attempt had been made to utilize the reaction principle to effect a local scour across an ocean bar similar to that existing in all concave bends of streams.

The writer, therefore, believed that, by a radical departure from the existing practice, there were numerous localities where a single structure of proper form and position might be made to utilize all the available forces of Nature to assist in creating a better channel across ocean bars and fulfill all the requirements as set forth in the first portion of this paper, but as no precedent existed and he must needs establish the correctness of the theory, applications were made in January, 1888, to the Board of Engineers, U. S. A., for a report on his plans, followed by requests to the Chiefs of Engineers, under dates of June 30th, July 30th and September 14th, 1888, for permission to make a demonstration, at a site to be agreed upon. These requests were never even acknowledged, although the letters were doubtless received, since they were not returned, while the Board made an adverse report to the Chief of Engineers on March 16th, 1888, concluding with these words: The Board finds that Professor Haupt's views "are purely

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"Fire Island Inlet: Is it Moving East or West?" *Journal* of the Franklin Inst., April, 1889.

"Report on the Harbor of Wilmington, Del." March, 1888.

"Jetties for Improving Estuaries." *Journal* of the Franklin Inst., April, 1888.

"Harbor Bar Improvements." *Journal* of the Franklin Inst., July, 1889.

"Galveston Harbor Problem." *Journal* of the Franklin Inst., Oct., 1891.

"How to Obtain Deep Water on the Gulf." Pub. Doc. S. 1583, 51st Congress, 1st Session, Jan., 1891.

"Plans for Improvement of Entrance of Rio Grande do Sul, Brazil," 1888.

"Littoral Movement of the New Jersey Coast, with Remarks on Beach Protection and Jetty Reaction." *Transactions*, Am. Soc. C. E., Sept., 1890.

(Most of the papers cited in this list will be found in the library of the Society.)

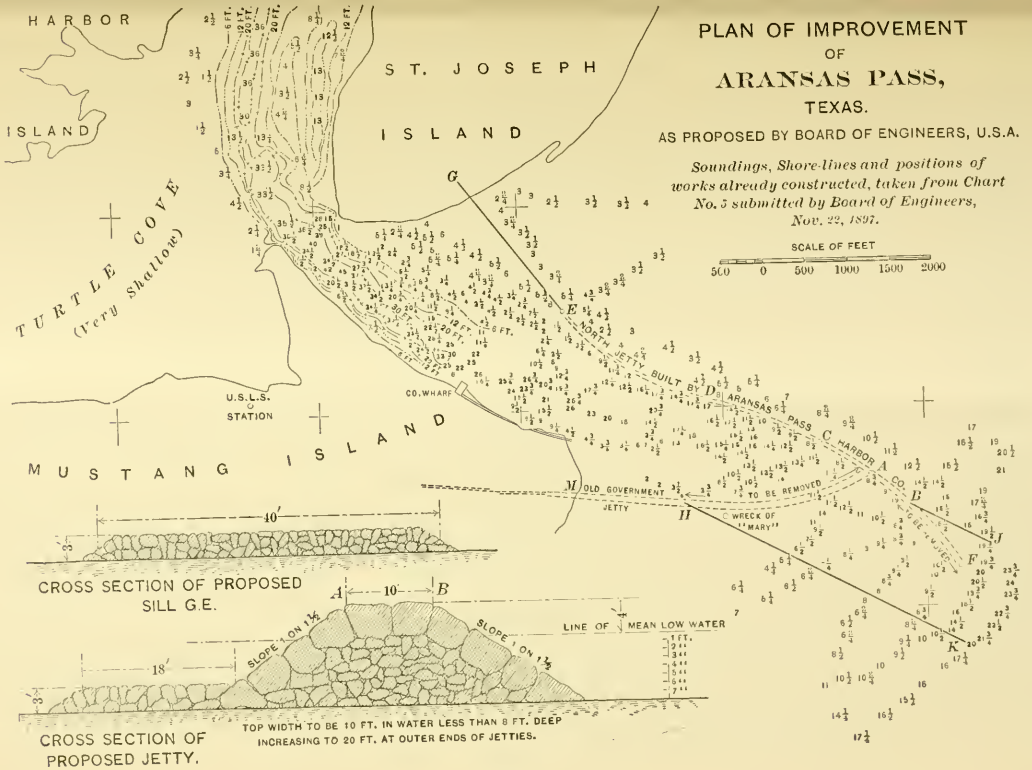
PLAN OF IMPROVEMENT
OF
ARANSAS PASS,
TEXAS.
AS PROPOSED BY BOARD OF ENGINEERS, U.S.A.
Soundings, Shore-lines and positions of works already constructed, taken from Chart No. 5 submitted by Board of Engineers, Nov. 22, 1897.
SCALE OF FEET
500 0 500 1000 1500 2000
HARBOR
ISLAND
ST. JOSEPH
ISLAND
TURTLE COVE
(Very Shallow)
U.S.L.S. STATION
CO. WHARF
NORTH JETTY BUILT BY ARANSAS PASS HARBOR CO.
OLD GOVERNMENT JETTY TO BE REMOVED
WRECK OF "MARY"
MUSTANG ISLAND
CROSS SECTION OF PROPOSED SILL G.E.
CROSS SECTION OF PROPOSED JETTY.
LINE OF MEAN LOW WATER
TOP WIDTH TO BE 10 FT. IN WATER LESS THAN 8 FT. DEEP INCREASING TO 20 FT. AT OUTER ENDS OF JETTIES.

FIG. 1.

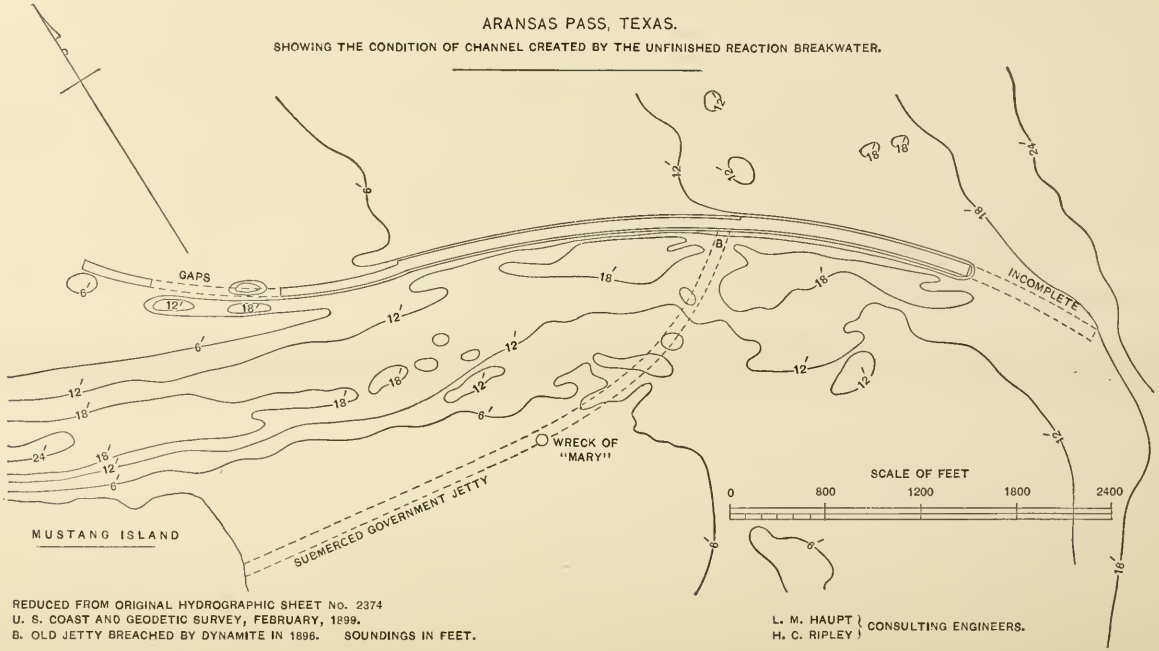
ARANSAS PASS, TEXAS.
SHOWING THE CONDITION OF CHANNEL CREATED BY THE UNFINISHED REACTION BREAKWATER.
GAPS
INCOMPLETE
WRECK OF "MARY"
SUBMERCED GOVERNMENT JETTY
MUSTANG ISLAND
SCALE OF FEET
0
800
1200
1800
2400
REDUCED FROM ORIGINAL HYDROGRAPHIC SHEET No. 2374
U. S. COAST AND GEODETIC SURVEY, FEBRUARY, 1899.
B. OLD JETTY BREACHED BY DYNAMITE IN 1896. SOUNDINGS IN FEET.
L. M. HAUPT } CONSULTING ENGINEERS.
H. C. RIPLEY }

FIG. 2.

theoretical, are unconfirmed by experience and contain nothing not already well known, which has a useful application in the improvement of our harbors."

The writer only learned of the existence of this report by accident at a later date, when it was quoted against his plans. He then discussed the report, forwarding copies thereof to the members of the Board and the then Chief of Engineers, but received no reply. It was therefore impossible to "confirm the theory by experience," and there was no alternative but to await a favorable opportunity, which did not arrive until 1895, when, everything else having failed at Aransas Pass, where over $500 000 had been spent by the Government in futile efforts to secure 20 ft. of water, a private party finally concluded to build half of the reaction breakwater on a predicted depth of 15 ft. for this part of the work. This depth has now been exceeded, but only after a breach was made in the obstructing Government jetty, officially reported to have disappeared prior to the beginning of the work. The partially controlled currents have actually removed about 400 000 cu. yds. of compact sand from the bar without the aid of a dredge or other mechanical appliance, and have prevented the deposition in the channel of a much larger volume driven along the coast by the angular wave movements.

The theory, therefore, which has been successfully applied at this admittedly difficult and unpromising location, was to protect the bar crossing near its weakest point from the drifting sand by a resisting mass of rock so placed as to develop a continuous reaction across the bar by the ebb currents, preventing their expanding over the entire external sector and conserving the energy of the effluent stream until it reached the deep water of the Gulf.

As means to this end the breakwater is detached from the shore to permit the tidal compartments of the inner bays to be filled freely by each flood; it is also composed of curves whose radii and centers are adjusted to the site in such manner as to cause deposits on the outer side of the structure, thus re-enforcing it, and scour on the inner side, where an excess of foundation material revets the slope, and a peculiar order of construction is adopted, whereby the scour is assisted by gravity and the advance of the bar seaward is prevented.

All these results appear to have been fully secured by the work thus far done, as is shown by the Coast Survey report of February, 1899. Modifications, however, are always to be expected in depths due to variations in the natural forces, but the progressive deepening created by the currents gives every assurance of their ability to maintain the channel they have already created and which would be still further deepened by the completion of the work as designed.

The modifications submitted to the last Congress by two members of the Board of Engineers in their report of December 17th, 1898, are

shown in Fig. 1, from which it appears that it was proposed to return, as nearly as might be, to the old-time method of two, nearly parallel, jetties and the closure of the beach entrance, supplemented by dredging at a cost of over $1 500 000. This plan, if adopted, would destroy the effective energy of the currents and prevent the completion of the demonstration of the principles which have been applied thus far at this entrance with unprecedented success, and would further involve an annual expense for maintenance, which is not now required.

This proposition, however, was rejected by Congress, which, as stated, made an appropriation of $60 000 to resume that portion of the old Government jetty which crossed the channel and provided that no portion of the reaction breakwater should be disturbed. This work was recommended by the United States District Engineer, June 8th, 1899, and was authorized July 11th, 1899.

As a part of the evolution of the improvement of ocean bars and in confirmation of the theory, the writer desires to add that independent efforts were being made by others in the same direction and with substantially the same conclusions as to the primary importance of arresting the littoral drift and of concentrating the scouring force by a single curved breakwater instead of controlling the ebb movements by means of two jetties. As early as 1883, Mr. H. C. Ripley, then United States Assistant Engineer to Colonel S. M. Mansfield, stationed at Galveston, Tex., designed a concave, detached breakwater for that entrance, to be placed on the windward side of the channel, which was approved by Colonel Mansfield and recommended to the Chief of Engineers for adoption, in a letter dated March 19th, 1884. This design is shown in the Report of the Chief of Engineers for 1884, opposite page 1300, and designated: "Site of North Jetty, proposed by Colonel Mansfield."

This coincidence, in connection with Mr. Wisner's work, both at the South Pass and at the mouth of the Brazos, led to the formation of the Board of Consulting Engineers for Aransas Pass by Mr. Brewster Cameron, which consisted of H. C. Ripley, George Y. Wisner and the writer. Their predictions have been fully verified by the results secured by natural scour after the partial removal of the unexpected obstructions.

A propos of the severe floods in the basin of the Brazos River, Texas, in July, it was expected that the larger volume discharging with greater velocity over the bar at its mouth between parallel jetties would materially improve the depth. On the contrary, the soundings showed but 15 ft. in several places, thus demonstrating the relative superiority of the principle of reaction by a single jetty, as applied at Aransas, to that of concentration by twin jetties at the Brazos River bar.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

TEST OF A MECHANICAL FILTER.

By EDMUND B. WESTON, M. Am. Soc. C. E.

TO BE PRESENTED NOVEMBER 1ST, 1899.

The object of this paper is to describe briefly the results of a three-months' test of a mechanical filter, which has recently been installed for the East Providence Water Company, at East Providence, R. I.

The New York Filter Manufacturing Company, of New York City, furnished the filter, which is of a type known as the Jewell Gravity Filter.

The entire work, including the filter house and pure-water well, shown on Plate VIII, Figs. 1 and 2, was designed and built under the direction of the writer, who acted as Consulting Engineer for the New York Filter Manufacturing Company.

The average daily quantity of water furnished to its consumers by the East Providence Water Company, at the present time, is about 200 000 galls. The available daily capacity of the filter is 500 000 galls., and the rate of filtration 125 000 000 galls. per acre per 24 hours. The filter house, pure-water well and pipes are arranged for a future addition of three more filters of the same design and capacity, when demanded by the requirements of the service.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in *Transactions*.

FIG. 1.

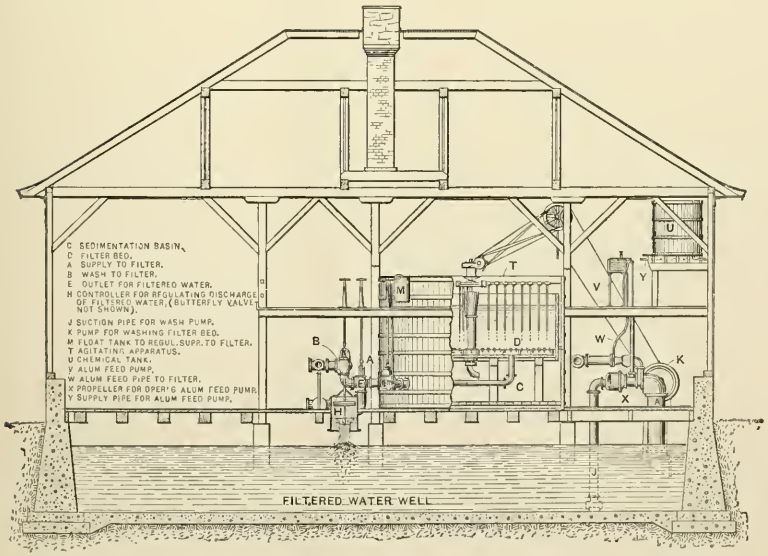
C SEDIMENTATION BASIN.
D FILTER BED.
A SUPPLY TO FILTER.
B WASH TO FILTER.
E OUTLET FOR FILTERED WATER.
H CONTROLLER FOR REGULATING DISCHARGE OF FILTERED WATER, (BUTTERFLY VALVE NOT SHOWN).
J SUCTION PIPE FOR WASH PUMP.
K PUMP FOR WASHING FILTER BED.
M FLOAT TANK TO REGUL. SUPP. TO FILTER.
T AGITATING APPARATUS.
U CHEMICAL TANK.
V ALUM FEED PUMP.
W ALUM FEED PIPE TO FILTER.
X PROPELLER FOR OPERG ALUM FEED PUMP.
Y SUPPLY PIPE FOR ALUM FEED PUMP.
FILTERED WATER WELL

FIG. 2.

The filter was run throughout the test under ordinary working conditions, and the filtered water was pumped directly into the mains and supplied to the consumers. The filter was in charge, under the writer's direction, of the regular pumping engineer of the East Providence Water Company. The writer gave directions from time to time, but did not, on the average, visit the filter plant more than once a week during the test.

The chemical analyses were made by Professor John Howard Appleton, of Brown University. The bacteriological work was done by Dr. Gardner T. Swarts, Secretary of the State Board of Health of Rhode Island, who also determined the color and alkalinity of the samples.

The samples of water for analysis were collected by the pumping engineer at about 8 o'clock in the morning, the filter-bed generally being washed about two hours earlier.

In a report, dated March 12th, 1894, describing experiments with experimental filters, made under the writer's direction, in Providence, from February, 1893, to January, 1894, inclusive, the writer's conclusions are given to the effect that water can be as satisfactorily purified by first-class mechanical filtration as by slow sand filtration.

Since this report was published, elaborate investigations, made with mechanical filters, at Louisville, Pittsburg and Cincinnati, with waters widely different from the water supply of Providence, have practically substantiated the writer's conclusions, as given in the Providence report. It would seem, therefore, that after taking into consideration the additional experimental results given in this paper, which were obtained with a filter absolutely in practical service, as though there could no longer be any reasonable doubt, if such may have existed, in regard to the practicability and efficiency of mechanical filtration, and that henceforth, in the broad field of water purification, mechanical filtration can be looked upon as being equally as desirable as slow sand filtration.

The chemical used during the test was sulphate of alumina, which was added to the raw water in the form of a coagulant solution, prepared by dissolving one part of sulphate of alumina in about 20 parts of filtered water. The solution was always thoroughly mixed before being used.

The theory of mechanical filtration, when sulphate of alumina has been added to the filtered water, may be described briefly as follows: The alumina causes an artificial precipitation; a portion of the alumina is decomposed, forming sulphates of other bases and a flocculent precipitate of aluminic hydrate. A portion of it also combines directly with the organic matter in the water, coagulating the same and thus helping to increase the precipitation. The degree of color of the water is also largely reduced by the uniting of the precipitated aluminic hydrate with the coloring matter in the water.

The filter was first put in service on February 25th, 1899, and it has been in regular operation since that time. The test was commenced on March 13th, which was as soon as a known grade of sulphate of alumina could be procured. Previous to March 13th, sulphate of an inferior grade was used, and was added to the raw water at rates of from $\frac{1}{2}$ to $\frac{3}{4}$ grain per gallon. This was the only quality of sulphate of alumina which could be purchased in Providence at the time, and the percentage of $Al_2 O_3$ which it contained was not known.

The sulphate of alumina used during the test and special experiments, contained about 22% of $Al_2 O_3$, with the exception that during one special experiment an inferior and cheaper grade containing about 17.53% of $Al_2 O_3$ was used.

The sulphate of alumina was added to the raw water at the rate of 1 grain per gallon, with the exception that in three special experiments $\frac{3}{4}$, $\frac{11}{16}$ and $\frac{3}{4}$ grain per gallon were used.

Tables Nos. 1, 2, 3 and 4 give all the results obtained during the test, with the exception of those of the special experiments, which are given in Table No. 5.

As can be seen by Tables Nos. 2, 3 and 4, the investigations relative to the chemical constituents and the color and alkalinity of the samples were concluded on May 31st.

It is the intention to continue the bacteriological analyses, at least until June 30th, 1899, and Table No. 1 gives all the results which have been obtained in time to be made use of in this paper, with the exception of those of the special experiments.

Fig. 1, Plate VIII, is a view of the filter house, which is located adjacent to the pumping station of the East Providence Water Company, at "Hunt's Mills." On the right of the plate can be seen a small portion of one end of the pumping station. The power required

for driving the agitator and wash pump is furnished by a turbine wheel located in the basement of the pumping station, and is transmitted to the filter house as shown on the plate.

Fig. 2, Plate VIII, is a sectional elevation through the filter, filter house and pure-water well. The filter and auxiliaries comprise a sedimentation basin, a crushed quartz filter-bed having an area of about 176 sq. ft., a pump for adding the coagulant to the raw water, an automatic controller connected to the main discharge pipe of the filter for maintaining a constant rate of filtration, a pump and appliances for washing and agitating the filter-bed and for washing out the sedimentation basin, and screens connected to collecting pipes at the bottom of the filter-bed. The raw water, to which the sulphate of alumina has been added, enters the sedimentation basin through the valve on the supply pipe at *A*, and is deflected by a curved casting in such a manner that it is caused to circulate slowly around the basin. The water rises from the sedimentation basin through the central pipe shown in Fig. 2, Plate VIII, to the required height above the filter-bed. The water passes downward and outward through the filter-bed, screens and collecting pipes to the main discharge pipe and controller, during the process of filtration, and inward and upward through the collecting pipes, screens and filter-bed, when the filter-bed is being washed. The screens prevent the quartz or any foreign substances from entering the collecting pipes and passing off with the filtered water.

The depth of the crushed quartz filter-bed is about 3.67 ft. The sedimentation basin has a capacity equal to a flow of about 17 minutes when the filter is being operated at the normal rate of 125 000 000 galls. per acre per 24 hours.

The coagulant pump is made of vulcanized rubber, and consists of six hollow arms radiating from a chambered hub, and bent in the direction of rotation. It is actuated by a propeller, situated in the main supply pipe, by the aid of an upright shaft and bevel gears. The pump gives excellent satisfaction, and its displacement is remarkably accurate.

The automatic controller is of a new and original design, and is a decided success. A careful test of the controller was made when it was first put in service, the flow of water being gauged accurately, and it was found, during a five-hour run of the filter, that the vari-

ation in the rate of filtration between the commencement and the end of the run was less than one-half of 1 per cent.

The manner in which the filter is operated is as follows: At the end of a run, or immediately before starting the filter, the filter-bed is washed thoroughly by forcing up through the screens and filter-bed a reverse flow of filtered water under pressure, the mechanical rake or agitator being operated at the same time, which adds materially to the efficient cleansing of the filter-bed. The water is forced up through the bed and the agitator is kept in motion until the water flowing from the overflow drain pipe is as clear as it was before being used for washing the filter. The necessary valves are then operated, and the unfiltered water, to which the sulphate of alumina is being added, is turned on the filter. The sedimentation basin is washed out by allowing the wash water at the top of the filter-bed to pass down into the basin through the central pipe shown in Fig. 2, Plate VIII, and thence out through a waste pipe at the bottom of the basin. The agitator shaft runs through the central pipe and carries at its lower end a curved nozzle. As the agitator shaft is revolved, the nozzle is given a circular motion, and the rapid current of water passing through it is thrown to all parts of the sedimentation basin and stirs up and flushes out the accumulated sediment. The sedimentation basin is also provided with a manhole.

The averages of the results of Tables Nos. 1, 2 and 3 show that by the process of filtration there was:

99.20% less bacteria in the filtered water than in the raw water;
6 % less total solids in the filtered water;
1 % less chlorine in the filtered water;
61 % less ferric oxide in the filtered water;
38 % less aluminic oxide in the filtered water;
29 % less free ammonia in the filtered water;
63 % less albuminoid ammonia in the filtered water;
83 % less color in the filtered water;
20 % increase of hardness in the filtered water.

Table No. 4 shows that in every instance the filtered water was more or less alkaline, and, consequently, that the raw water was sufficiently alkaline to more than decompose the 1 grain per gallon of sulphate of alumina added to it.

TABLE No. 1.—Bacteriological Analyses of Samples, by Dr. Gardner T. Swarts.

Results obtained during the time that 1 grain of sulphate of alumina per gallon was used,

Rate of filtration, 125 000 000 galls. per acre per 24 hours.

Date.	Bacteria per Cubic Centimeter.		Percentage of Reduction.	Date.	Bacteria per Cubic Centimeter.		Percentage of Reduction.
	Raw Water.	Filtered Water.			Raw Water.	Filtered Water.	
Mar. 13.....	768	4	99.49	Apr. 27.....	422	9	97.87
" 14.....	595	5.5	99.08	" 28.....	280	2.5	99.11
" 15.....	Steri lized	fil ter-bed.		" 29.....	370	6	98.38
" 16.....	1 299	9	99.31	May 8.....	266	9	96.62
" 17.....	1 257	7	99.45	" 9.....	976	3	99.69
" 18.....	683	4	99.41	" 10.....	708	13.5	98.09
" 31.....	782	1.5	99.82	" 11.....	150	5	96.66
Apr. 1.....	499	7	98.60	" 12.....	466	3.5	99.25
" 3.....	636	1.5	99.76	" 13.....	305	4	98.69
" 4.....	628	2	99.68	" 15.....	225	1	99.56
" 5.....	545	4	99.27	" 16.....	238	0.5	99.79
" 6.....	855	3	99.65	" 17.....	306	0.5	99.83
" 7.....	1 910	19	99.01	" 18.....	473	0	100.00
" 8.....	1 009	6.5	99.36	" 19.....	210	0.5	99.76
" 10.....	1 175	6.5	99.45	" 20.....	228	1	99.56
" 11.....	943	9.3	99.01	" 22.....	238	0.5	99.79
" 12.....	1 443	9	99.38	" 23.....	279	1	99.64
" 13.....	336	4.3	98.73	" 24.....	228	1	99.56
" 14.....	Lost.	4	.....	" 25.....	275	0	100.00
" 15.....	798	1.6	99.84	" 26.....	270	0.5	99.81
" 17.....	765	7.5	99.02	" 27.....	185	1	99.46
" 18.....	578	1.5	99.74	" 29.....	454	4.5	99.01
" 19.....	865	11	98.73	" 30.....	334	11.5	96.56
" 20.....	546	3	99.45	" 31.....	458	10	97.82
" 21.....	699	2	99.71	June 8.....	331	4	98.79
" 22.....	499	3	99.40	" 9.....	494	5.6	98.86
" 24.....	293	3	98.98	" 10.....	341	6.3	98.15
" 25.....	697	0.5	99.93	" 12.....	354	0.3	99.92
" 26.....	724	11	98.48	" 13.....	243	2	99.18
				Averages.....	570	4.5	99.20

Special experiments were made from March 20th to 30th, from May 1st to 6th and from June 1st to 7th.

The bacteria in the different samples were cultivated during periods of from five to six days.

Special attention is also called to the fact that Table No. 1 shows that the average number of bacteria per cubic centimeter found in the samples of filtered water is less than 5, and that the above summary shows that the filtered water contained 38% less alumina than did the raw water before the sulphate of alumina was added to it.

The results of the special experiments mentioned are shown in Table No. 5.

TABLE No. 2.—Chemical Analyses of Samples, by Professor John Howard Appleton.

Results obtained during the time that 1 grain of sulphate of alumina per gallon was used.

Rate of filtration, 125 000 000 galls. per acre per 24 hours.

The numbers express parts (by weight) in one million parts of water (by weight).

Date.	Total solids.	Total hardness.	Chlorine.	Ferric oxide.	Aluminic oxide.	N as free ammonia.	N as albuminoid ammonia.	N as nitrates.	N as nitrites.
				Raw Water.					
Apr. 6.....	39.0	16.0	4.8	0.58	0.47	0.04	0.22	0.60	Trace.
" 13.....	39.3	14.0	7.0	0.60	0.80	0.10	0.26	0.90	"
" 20.....	39.9	17.0	6.4	0.61	1.05	0.05	0.26	0.70	"
" 27.....	43.7	18.0	6.2	1.00	0.75	0.02	0.22	0.70	"
May 11.....	53.1	21.0	6.4	0.91	1.84	0.03	0.38	0.60	0
" 18.....	54.9	21.0	6.1	1.01	0.34	0.03	0.34	0.60	Trace.
" 25.....	49.6	20.0	6.4	1.09	0.76	0.04	0.32	0.60	"
Averages.....	45.6	18.1	6.2	0.83	0.86	0.04	0.29	0.67	.....
				Filtered Water.					
Apr. 6.....	38.9	22.0	4.8	0.61	0.44	0.04	0.10	0.60	Trace.
" 13.....	40.4	19.0	6.0	0.45	0.55	0.05	0.11	0.70	"
" 20.....	37.4	22.0	6.4	0.28	1.02	0.05	0.10	0.60	"
" 27.....	39.6	19.0	6.2	0.19	0.26	0.01	0.07	0.60	"
May 11.....	47.2	23.0	6.3	0.20	0.55	0.02	0.14	0.40	0
" 18.....	49.8	23.0	5.8	0.13	0.47	0.03	0.12	0.50	Trace.
" 25.....	46.5	24.0	7.2	0.40	0.45	0.02	0.13	0.60	"
Averages.....	42.83	21.7	6.1	0.32	0.53	0.03	0.11	0.57	.....
				Average Percentage, More or Less, in the Filtered Water.					
	6% —	20% +	1% —	61% —	38% —	29% —	63% —	15% —	.....

Special experiments were made from March 20th to 30th, and from May 1st to 6th.

As may be noticed, the percentage less of bacteria from May 1st to 6th, when $\frac{3}{4}$ of a grain of sulphate of alumina per gallon was used, is considerably less than it was from March 20th to 25th, when the same quantity of sulphate of alumina was being used. The difference may be accounted for partially by the fact that the average number of bacteria in the raw water was nearly five times as large from March 20th to 25th, when the more favorable results were obtained, than it was during the period of less favorable results, from May 1st to 6th; but the principal cause is due to the sample of filtered water of May 4th having contained a larger number of bacteria than was found in any

sample during the test, viz., 61, and as the sample of raw water of this date contained but 298, the percentage of reduction was only 78.89; which, being averaged up with the percentage of the other five days, brought down the total average of the special test, from May 1st to 6th inclusive, to 94.03 per cent.

The results given in Tables Nos. 1, 2, 3 and 4, are remarkably satisfactory; but on account of several instances in Table No. 1, when the bacterial reductions are shown to be below 98%, it may be well to state that the filter has been working at a disadvantage for a comparison by percentages, on account of the small number of bacteria in the raw water.

TABLE No. 3.—Color of Samples. Determined by Dr. Gardner T. Swarts.

Results obtained during the time that 1 grain of sulphate of alumina per gallon was used.

Rate of filtration, 125 000 000 galls. per acre per twenty-four hours.

Date.	Raw water.	Filtered water.	Date.	Raw water.	Filtered water.	Date.	Raw water.	Filtered water.	Percentage of color removed.
Mar. 13	0.30	0.06	Apr. 12	0.50	0.10	May 11	1.00	0.10	.....
" 14	0.30	0.06	" 14	0.50	0.10	" 12	1.00	0.10	.....
" 15	Sterilized filter-bed.		" 15	0.50	0.10	" 13	0.90	0.10	.....
			" 17	0.50	0.10	" 15	0.80	0.10	.....
" 16	0.30	0.06	" 18	0.60	0.10	" 16	0.90	0.10	.....
" 17	0.30	0.06	" 19	0.60	0.10	" 17	0.80	0.10	.....
" 18	0.30	0.06	" 20	0.60	0.10	" 18	0.80	0.10	.....
" 31	0.40	0.10	" 21	0.60	0.10	" 19	0.70	0.10	.....
Apr. 1	0.40	0.10	" 22	0.60	0.10	" 20	0.70	0.10	.....
" 3	0.40	0.10	" 24	0.60	0.10	" 22	0.70	0.10	.....
" 4	0.40	0.10	" 25	0.60	0.10	" 23	0.60	0.10	.....
" 5	0.40	0.10	" 26	0.60	0.10	" 24	0.60	0.10	.....
" 6	0.40	0.10	" 27	0.70	0.10	" 25	0.70	0.10	.....
" 7	0.40	0.10	" 28	0.70	0.10	" 26	0.60	0.10	.....
" 8	0.40	0.10	" 29	0.60	0.10	" 27	0.60	0.10	.....
" 10	0.50	0.10	May 8	0.70	0.10	" 29	0.60	0.10	.....
" 11	0.40	0.10	" 9	0.70	0.10	" 30	0.60	0.10	.....
" 12	0.50	0.10	" 10	0.70	0.20	" 31	0.50	0.10	.....
						Average	0.58	0.10	83.0

Special experiments were made from March 20th to 30th and from May 1st to 6th.

As can be seen by Table No. 1, the average number of bacteria found in the filtered water is less than 5 per cubic centimeter, which is a remarkably small number (the average number found in the effluent of the filter of the Lawrence water-works during March and April, 1898, being 28). Therefore, if the results given in Table No. 1

should be used for comparison with results obtained with other filters, the small number of bacteria found in the filtered water should be duly taken into consideration, as well as the percentages of reduction.

TABLE No. 4.—Alkalinity of Samples. Determined by Dr. Gardner T. Swarts.

Results obtained during the time that 1 grain of sulphate of alumina per gallon was used.

Rate of filtration, 125 000 000 galls. per acre per 24 hours. The alkalinity is expressed as calcium carbonate in parts per 1 000 000.

Date.	Raw water.	Filtered water.	Date.	Raw water.	Filtered water.	Date.	Raw water.	Filtered water.
Mar. 13	5.5	1.7	Apr. 13	10.2	Lost	May 11	14.0	6.0
" 14	7.5	2.0	" 14	11.5	4.0	" 12	14.5	6.5
" 15	Steri lized filter-beds		" 15	11.5	5.0	" 13	14.5	5.5
			" 17	12.0	4.5	" 15	15.0	5.5
" 16	6.7	1.5	" 18	11.0	4.0	" 16	14.0	5.0
" 17	7.0	2.0	" 19	11.0	4.5	" 17	14.5	6.5
" 18	6.7	2.0	" 20	12.0	2.0	" 18	14.5	6.0
" 31	9.0	2.7	" 21	11.0	4.5	" 19	14.0	6.0
Apr. 1	8.5	3.0	" 22	8.5	4.7	" 20	14.5	6.0
" 3	9.2	2.7	" 24	14.0	7.0	" 22	14.0	8.0
" 4	9.5	3.2	" 25	14.0	7.0	" 23	14.0	6.0
" 5	8.7	3.2	" 26	14.0	6.0	" 24	14.0	7.0
" 6	6.5	3.0	" 27	14.0	6.0	" 25	14.0	7.0
" 7	11.0	3.7	" 28	14.5	6.0	" 26	15.0	6.0
" 8	10.0	3.2	" 29	13.5	6.0	" 27	14.5	5.0
" 10	9.0	3.7	May 8	15.0	6.5	" 29	14.0	5.0
" 11	10.5	3.2	" 9	14.5	6.0	" 30	14.0	5.0
" 12	11.0	4.0	" 10	15.0	6.0	" 31	14.5	6.0

Special experiments were made from March 20th to 30th and from May 1st to 6th.

TABLE No. 5.—Results of Special Experiments.

Date. (Inclusive.)	Sulphate of alumina used. Grains per gallon.	Number of Bacteria.		Percentage less bacteria in filtered than in raw water.	Percentage less color in filtered than in raw water.
		In raw water.	In filtered water.		
March 20th to 25th.	0.75	1 768	28	98.32	79
March 27th to 30th.	0.60	875	9	98.75	73
May 1st to 6th.....	0.75	360	19	94.03	75
June 1st to 7th.....	1.00*	640	15	96.82	..

* Sulphate of alumina of low grade, containing 17.53% of $Al_2 O_3$.

Experiments have shown that some species of bacteria will multiply even in distilled water that has been sterilized; and it is quite possible that, should sterilized water be applied to the filter instead of the raw water, a few bacteria might be found in the effluent as it flowed from the filter, as it would probably be impossible to keep any practical

filtering medium, or water which had been exposed to the atmosphere, completely sterile during the process of filtration.

The disadvantage under which the filter has been working is in accordance with the supposition that a few bacteria may, at times, grow in a filter and be carried through it during the process of filtration. These few bacteria would, in ordinary practice, be counted among others found, if there were such, in the filtered water. Now, if there were 2 000 bacteria per cubic centimeter in the raw water, and a small number per cubic centimeter should grow in the filter, they might not appreciably affect the percentage of reduction, owing to their number being relatively very small in comparison with the number in the raw water. If, however, the number of bacteria that grew in the filter was the same, and there should be 200 or 300 per cubic centimeter, for instance, in the raw water, instead of 2 000, the percentage of reduction might be affected considerably, as the small number that grew in the filter might bear an appreciable proportion to the 200 or 300 in the raw water.

The cost of operating the filter since it was first put in service has been practically the cost of the sulphate of alumina used, as no additional labor has been required, other than that already employed at the pumping station. The cost of operating, therefore, based upon 1 grain per gallon, and, considering the best grade of sulphate of alumina used during the test, would be $2.15 per 1 000 000 galls. of water filtered.

The total cost of the filter plant (shown on Plate VIII) is estimated to be about $11 500. If the three additional filters, for which the filter house was designed, should be added, as has been mentioned previously, at the same cost per filter as the one which has been installed, the cost of the completed plant, representing a capacity of 2 000 000 galls. per 24 hours, would be about $21 000, or at the rate of $10 500 per 1 000 000 galls.

The East Providence Water Company is more than satisfied with the filter plant, and the customers of the company are much pleased with the filtered water, the appearance of which is practically the same as that of distilled water.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

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COMPARATIVE TESTS OF BITUMINOUS STEAM COALS.

Discussion.*

By Mr. JOHN W. HILL.

Mr. Hill. JOHN W. HILL, M. Am. Soc. C. E. (by letter).—It should be understood that these tests, primarily, were to determine the relative physical value, under uniform conditions of service, of the steam coals available in the Cincinnati market, especially for use in the City Water Department, and the only way in which this could be accomplished satisfactorily was by firing the coals in the Department boilers by the Department firemen. To avoid interference with the regular daily operation of the Water-Works, the Board of Administration required that the tests should be made at the Hunt Street Pumping Station, which has at all times a small surplus of power. It required that the coal should be fired in boilers which had no patented appliances especially adapted to one class or kind of coal, and that the steam should be used in performing the daily work of the pumping station. The author has no doubt that the Galloway boilers were not the best for all the coals tested, but believes that they were the best which could be had at the Hunt Street Pumping Station. With reference to the fitness of the boilers for this work, Mr. Kent evidently has overlooked the following paragraph on page 178:

"It is quite possible that the boilers used in these tests were not the best for all the coals tested, and that some kinds and grades of coal suffered accordingly; but that is a condition which perhaps will

* Continued from August, 1899, *Proceedings*. See April, 1899, *Proceedings*, for Paper by John W. Hill, M. Am. Soc. C. E., on the subject.

always be found true, for no form of boiler and furnace can be equally well adapted to every variety of bituminous (and semi-bituminous) coal, nor could changes be made in the boiler and furnace, and in the mode of firing, for the different kinds of coal, without raising a suspicion in the investigator's mind that by so doing he would be favoring one coal at the expense of another." Mr. Hill.

And on page 200:

"Reference to the physical efficiencies of these coals (Table No. 23) indicates that none of the coals was giving very high results, and some, undoubtedly, were worked at a disadvantage, although the evidences of this were not apparent at the time; but high efficiency of the low-grade coals cannot always be attained except in furnaces especially adapted for them. All the losses of heat by chimney draft, etc., should be limited to 30% of the ultimate heat value of the coal; and in designing boilers and furnaces, due regard should be had for the work to be performed and the coal to be used."

If the author understands Mr. Kent correctly he suggests that these different coals should have been tried under different boilers and by different firemen. Had this been done, it would have been impossible to have formed any correct estimate of the influence of boilers and firemen on the several coals, and would wholly have failed to answer the question submitted to the author by the Board of Administration. Undoubtedly, the proper method for the determination of the ultimate value of each coal would have been to test it with several kinds of boilers and furnaces, but this was wholly outside the scope of the author's authority or opportunities in these investigations.

Mr. Kent seems at first to commend the internally-fired boilers used in these trials, and then to object to them, because the furnace was not lined with fire-brick to secure better combustion of the volatile matter in the coals. This suggests the fire-box boilers for some kinds of coals, and boilers with brick furnaces for other kinds of coals; but since all these coals contained from 20 to 35% of volatile matter, any objection to the use of the fire-box boilers for the bituminous coals will apply, but possibly with less force, when used with semi-bituminous coals.

The author must take issue with Mr. Kent with reference to the rate of evaporation used in these tests. The Galloway boilers, and fire-box boilers generally, which have come under the author's observation, have been worked successfully at much higher rates than that adopted for these tests (5.33 lbs. of steam per square foot of heating surface per hour). Indeed, at the contract trial (from which some data are given on page 180), these boilers were worked at the rate of 7.57 lbs. of steam per square foot of water-heating surface per hour, from the temperature of feed water (193° Fahr.) to steam at 92.8 lbs. by gauge, or at a rate nearly 50% higher than that adopted by the author. Under this condition the boilers were said to have made, with Pittsburg Nut coal, 12.40 lbs. of steam from and at 212° Fahr. per pound of com-

Mr. Hill. bustible. The trial mentioned was made by the late Mr. George H. Sellers, of Wilmington, Del.

It will be observed that, from Mr. Kent's point of view, the coal (Pittsburg) used by Mr. Sellers, by reason of its high percentage of volatile matter, was peculiarly unfitted for fire-box boilers.

With reference to the rate of work, the author does not believe that these coals should be burned at lower rates than shown in Column 6 of Tables Nos. 13, 14 and 15. In fact, since the trials were made, it has been thought that some of the coals were burned at too low a rate, per square foot of grate surface per hour, to obtain the best effects. It will be remembered that the boilers were worked at substantially the same rate of evaporation per square foot of heating surface per hour for all coals, and the extreme variation from the average rate was, with one exception, within 5 per cent.

The extreme ground, assumed by Mr. Kent, that the boilers were in an indifferent condition, driven too hard, and fired by inexperienced firemen, is wholly unwarranted. The boilers were in excellent condition, regularly cleaned at the end of each fourth test, and the firemen were the best employed by the City Water Department, and possibly were as expert in handling these coals as any firemen who were available for these tests.

The figures quoted by Mr. Kent on page 322 do not indicate what he terms "variable firing," but do indicate the variable quality of coal from the same dealers, and, presumably, from the same mine. Variations of 5 or 6% in the quality of coal from different sources and entries of the same mine are well known. In fact, in a given seam of coal a marked variation in heating power may be found at different heights in the seam.

Considering the experimental work of Professors Lord and Haas,* which shows, by the Mahler calorimeter, extreme variations (on combustible) in different samples of coal from the same measures, the following percentages are found:

Pittsburg coal	5.3
Upper Freeport coal	5.5

Mr. Barrus, † from calorimetric tests of the same coals (on coal) obtained the following percentage variations:

George's Creek (Cumberland), coal	9.45
New River coal	4.20
Pocahontas coal	6.80
Clearfield Coal	8.20

The Acme coal and Coalburg coal are supposed to be from the same seam, and mined 12 miles apart. The difference between these coals

* *Loc. cit.*, page 188.
† *Loc. cit.*, page 188

by physical tests on combustible was 7.7 per cent. Again, the Cedar Grove and Belmont coals are mined by the same parties, on the same property, the seams being about 340 ft. (vertical) apart. The difference between these coals on combustible was 4.7 per cent. Part of these differences may be due to variable firing, but the larger portion is due to variability in the quality of the coal from different seams on the same property, or from the same seam on widely separated properties. Mr. Hill.

Mr. Kent's comparison of the coals from the Pittsburg mines, to demonstrate his belief that the firing in these tests was variable, is not altogether fair. The Brown mines and the Woods mine are separated about 20 miles, although supposed to be working the same seam of coal, and, in view of the reasonable uniformity of the several grades of coal from the respective mines, the comparison for variable firing (upon the assumption, which is not true, that the coal is of uniform quality throughout the seam) should be made upon the average performance of each coal, thus:

Source of Coal.	Pounds of steam per pound of Combustible from and at 212° Fahr.
Brown, four samples	9.42
Catsburg, four samples	9.46
Cincinnati, two samples	9.29
Woods, four samples	9.60
Monongah (West Virginia), one sample	9.58

All these coals are from the Pittsburg measures, and the combustible in the nut and slack is as good as in the higher grades of coal. Upon this showing, the extreme difference in the performance, and (by Mr. Kent's method of reasoning) variability of firing, is 3.23%; or, if the Cincinnati coal is omitted, the difference in these coals is less than 1.2 per cent. The Cincinnati coal, in both physical tests and by the calorimeter, falls distinctly under the other twelve samples from the Pittsburg seam.

The theory that all kinds of coal such as the author investigated will give a fairly uniform efficiency in any given type of boiler, is a novel proposition to the author, and is scarcely consistent with Mr. Kent's original suggestion, that these boilers were not well suited to all the coals tested. Experience has taught the author that the physical efficiency of a given coal will depend very much upon the adaptability of the boiler and furnace to that coal, and variations in efficiency must be anticipated when a single type of boiler and furnace is used with different kinds of coal.

Long before these investigations, the author had made many experiments on coals with the Thompson form of calorimeter, and had studied carefully all the available literature with reference to the use of the Berthelot bomb and other methods proposed for the determination of the heating power of fuels, and reached the conclusion that

Mr. Hill. if the heat value of any substance had been accurately determined by any form of calorimeter, it was that of carbon; and if the heating value of pure carbon burning without ash is now known, that the Carpenter instrument would furnish as accurate and reliable results when properly standardized as the Berthelot or any other form of bomb, and, therefore, he adopted it for these investigations.

In order to check his work as well as could be done from the available data, Table No. 20 was compiled. From this it appears that, with coal from a given locality, no startling differences occur between the work of the author and that of Messrs. Hale and Williams with a modified form of Berthelot bomb; of Professor Carpenter with his own instrument; and of Professors Lord and Haas with the Mahler bomb; and these differences are no greater than each experimenter probably has obtained in his own work.

The author sees no greater difficulty in standardizing the Carpenter calorimeter by pure carbon than in standardizing the Berthelot bomb by naphthalene, or any other substance of supposed known heat value. In one case, the heating power of pure carbon is assumed to have been accurately determined, and in the other case, the heating power of naphthalene is assumed to have been accurately determined. So far as combustion of the coal sample is concerned, there is no reason why it should not be as complete and perfect in the combustion chamber of the Carpenter instrument as in the combustion chamber of the bomb; nor does it appear to the author any more difficult to account for the total heat of combustion in the Carpenter instrument than in the Berthelot or Mahler bombs.

The use, in excess, of pure oxygen under pressure insures perfect and complete combustion, thus avoiding the usual loss due to imperfect combustion of the volatile matter in coal, when burned under steam boilers, and insuring by the presence of pure oxygen, the conversion of all combustible matter into carbon dioxid, sulphuric acid and water. In using the Carpenter instrument, it is not convenient to collect the nitric acid obtained from the substance burned if nitrogen be present, nor is this necessary, because the heat liberated in converting the nitrogen into nitric acid seldom exceeds one-third of 1% of the total heat of the substance.

The advantage of the Carpenter over all other forms of coal calorimeter is in its simplicity of form and operation, and the readiness with which the notes may be reduced, thus avoiding the possible errors of complicated apparatus and tedious computation.

In reply to Mr. Kent's remark under Table No. 29, that in one instance (Cincinnati 3d Pool coal) the author's calorimetric work is not entitled to credit, Table No. 31, comparing the results obtained by the author, with the results obtained from the same kinds of coals by Professors Lord and Haas, and Professor Carpenter is offered.

TABLE No. 31.—B. T. U. Per Pound of Coal. Mr. Hill.

Kind of Coal.	Author.	Lord and Haas.	Carpenter.
New River	15 163	.....	.....
Pocahontas	15 133	14 906	15 094
Thacker	14 526	14 020	.....
Pittsburg (av.)	13 348	13 607	.....
Southern Ohio	12 733*	12 136†	.....
B. T. U. Per Pound of Combustible.			
New River	15 757	.....	.....
Pocahontas	15 456	15 750	15 870
Thacker	15 554	15 230	.....
Pittsburg (av.)	14 237	14 960	.....
Southern Ohio	14 218*	14 050†	.....

* Average Hocking Valley and Jackson coals.
† Hocking Valley coal.

The author does not know whether Mr. Kent considers the calorimetric work of the other investigators as entitled to credit, but if he does, it seems that the differences in heating value of the parallel coals tested are not so large as to suggest grave errors in the author's work. The extreme difference in the author's work on the Pittsburg (Pennsylvania) coals, by physical tests, is 3.23%, and by calorimeter (on combustible) 2.4%, which seems to indicate that Mr. Kent has made a mistake in rejecting the calorimetric value of Pittsburg-Cincinnati coal in his Table No. 29.

It may be observed in passing, that, in Table No. 29, Mr. Kent does not attempt to fit his curve to the Ohio coals having nearly the same percentage of fixed carbon in the net combustible as the Pittsburg and some of the Kanawha coals, doubtless for the reasons which he gives on page 326.

The heats given by Mr. Kent's curve for semi-bituminous coals containing about 80% of fixed carbon in the combustible, agree well with the results obtained by the calorimeter, but, for coals much lower in fixed carbon, the differences are too great to admit of using the curve for either practical or scientific purposes.

For more than half a century it has been held that the heating value of bituminous and semi-bituminous coals was directly proportional to the percentage of fixed carbon in the combustible matter, although there is a limit to the application of this theory, because it is well known that the heat energy per unit of weight is less for the anthracite coals than for the semi-bituminous coals, like New River, Cumberland and Pocahontas, and a point is reached in the ratio of the fixed carbon to volatile matter which indicates the highest heat value theoretically attainable in coal. Above this point or ratio of fixed carbon to volatile matter, the thermal value of the coal declines until it reaches pure carbon, with a heating value of 14 650 B. T. U. per pound, and below this the heating value declines in an increasing ratio until the low value of 12 200 B. T. U. per pound is reached, with a combustible of which 50% is fixed carbon and 50% volatile matter.

Mr. Hill. The author does not share Mr. Bryan's fear that serious errors in the amount of coal charged to each trial, respectively, might have arisen from the method of starting and stopping the trials, considering that each of the four furnaces was fired from four to five times per hour. If all the furnaces had been charged with green coal just before a trial was started, and the grates left bare at the end of the trial, the maximum error in the coal charged would have been 1½%; but with the fires on each of the four grates burned down to the lowest point, while maintaining the steam pressure at the beginning and end of each trial, the probable error in the weight of coal charged must have been much less than the usual errors of observation. During the last 30 minutes, just before starting and stopping each trial, the firemen were under constant supervision, in order to avoid an excess or deficiency of coal on the grates, while maintaining the steam pressure and the speed of the pumping engine.*

Considering the temperature at which the feed-water was of necessity measured,† the meters seem to be the best device for the purpose. Under other conditions than those described in the paper, the author agrees with Mr. Bryan that it is preferable to weigh the water to the boilers.

With reference to cleaning the boilers before Trial No. 50‡ the paper neglects to state that the boilers were cleaned inside and out, and the grates and bridge-wall thoroughly overhauled and made equal to new.

In view of the relative value of New River and Pocahontas coals, it is singular that the Navy Department has not accorded the former coal the rating which it deserves. The author understands, unofficially, that New River coal is rated at 90, Pocahontas coal being taken as the standard at 100; while, in fact, under the steam boilers by physical test, and upon the heating value by both coal and combustible, New River coal possesses a slight advantage over Pocahontas coal, and should be rated as high as the latter.

In burning all coals under steam boilers, only a fraction of the total heat goes into the water in the boilers. The remainder, which is lost for useful effect, is represented by the heat required to produce chimney draft, by evaporation of moisture in the air which passes through the furnaces and boilers, by evaporation of the moisture in the coal, by the latent heat in the gases of combustion, and by radiation and conduction of heat from the exposed surfaces of furnaces and boilers. Notes sufficient to show the losses of heat in detail were not taken during these tests, nor was it at the time thought necessary, because all the coals were burned in the same boilers, by the same firemen, and under uniform conditions of practice, and with "comparative" rather than "ultimate" effects in view.

* Refer to paragraph bottom of page 147 and top of page 148.
† Refer to paragraph bottom of page 148 and top of page 149.
‡ Refer to page 156. "Condition of Boilers."

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PAPERS AND DISCUSSIONS.

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PILE-DRIVING FORMULAS: THEIR CONSTRUCTION AND FACTORS OF SAFETY.

Discussion.*

By Messrs. FOSTER CROWELL, E. SHERMAN GOULD, HORACE J. HOWE and L. J. LE CONTE.

Mr. Crowell.

FOSTER CROWELL, M. Am. Soc. C. E.—The author's experiments upon the impact of falling weights, as recorded in the table on page 286, having been made with a spring-balance, the observed results were the resistances of the spring-balance, exerted through unrecorded distances of retardation of the falling weights, and therefore if the apparatus had happened to be of a different degree of sensitiveness the values would necessarily have been correspondingly different; consequently the results could not be taken as the absolute values of impact of the falling weights, to arrive at which it would be necessary to determine accurately, not only the resistances developed, but also the distances of retardation. The experiments, therefore, could not be accepted as establishing, even practically, the author's claim, upon which he bases his formula. For, whatever the weight of the impinging body, or the velocity of the impact, there is, in every case, a finite amount of work U, yielded upon the opposing resistances; there is in every such case a certain mean resistance R, overcome through a certain

* This discussion (of the paper by Charles H. Haswell, M. Am Soc. C. E., printed in the *Proceedings* for May, 1899) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to October 20th, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

Mr. Crowell. space S, in the direction in which that resistance acts, which resistance and space are such that $R\,S = U$, and $R = \frac{U}{S}$; if, therefore, the space S be exceedingly small as compared with U, there will be an exceedingly great resistance R, overcome through that small space, and *vice versa*. Taking as the values of R, the "impacts" of the table, we find that the corresponding values of S, in inches, are respectively 0.375, 0.410 and 0.461. Thus it is apparent that, had the apparatus been so constructed as to weigh with less movement, the recorded impact might have been very much higher; also, that the movement of the apparatus is analogous to a pile penetration of about 8 ins. under a 20-ft. fall of the ram, in the first case given, but to only about $5\frac{1}{2}$ ins. in the third case, which shows the influence of the friction and lost motion in the apparatus.

Mr. Haswell's formula, as presented, is limited to a penetration of $\frac{1}{2}$ in., and it would be interesting to know what provision the author intended to be made in cases where the observed penetration of the driven pile exceeded that amount, as such cases, in the speaker's experience, constituted the real test of reliability and usefulness of a pile-driving formula. The ideal formula, and the practical one as well, should contain within itself the elements of application to all the cases usually to be met with in practice, and on that account, as well as upon others, the speaker considered the Wellington form of formula preferable to that of Mr. Haswell.

The author has condemned the Wellington formula, including it with those which are arbitrary and at variance with accepted rules, and specifically because it "presents the exceptional condition of the weight being multiplied by twice the height of the fall," claiming that the true formula should be founded upon the actual *vis viva* of a body falling freely, as represented by its velocity, viz., $\sqrt{2\,g\,h}$. This is a misconception, both as to the fact and as to the theory of *vis viva*. The *vis viva* of a body falling freely being $M\,v^2$, and h being the height through which the body must fall to acquire the velocity, v, $v^2 = 2\,g\,h$; therefore $h = \frac{v^2}{2\,g}$, and $W h = \frac{1}{2}\,M\,v^2$; that is to say, the work accumulated in a body falling freely, available for extraneous effect, is expressed by half the *vis viva*, and is always equal to the simple product of its weight multiplied by the distance through which it has fallen. This effect may be expressed in foot-pounds, or inch-pounds, or half-inch-pounds, or by any other combination of weight and fall, but it can never by any possibility exceed itself, and is an immutable feature of the laws of falling bodies applicable to all heights.

Mr. Crowell.

Vis viva, according to Mosely,*

"Does not represent a pressure, or force, but the numerical equivalent of the product of a certain number of units of pressure and a certain number of units of path. The one magnitude being of as totally different order from the other as an area is different from a line, and therefore having no common unit of measure."

Impact and *vis viva* are thus totally different things. What enables the descending ram to drive the pile is the energy stored therein, but not all of that energy is available for the purpose, half of it being required to maintain the movement of the ram. That which is available is known as the accumulated work, to be expended upon the sum of the resistances offered by the pile.

In the Wellington formula the principle of accumulated work is the basis; it is expressed in inch-pounds, but as it contains a silent factor of safety of 6, and, as, for convenience, h is given in feet, the accumulated work is written $2\,W\,h$ instead of $W\,12\,h$. The denominator is the measure of resistance developed by the pile, as indicated by its penetration under the last blow, or settlement, s, also expressed in inch terms. Were it not for the presence of resistances in the pile-driving machine, and other causes tending to oppose the driving of the pile, s would be the full measure of the net resistance, and the quotient would be that resistance; but these various unmeasured opposing forces must be taken into consideration, and they are represented by the constant quantity, 1, in the denominator.

Thus we have the derivation of the Wellington formula,

$$Q = \text{Safe Load} = \frac{2\,W\,h}{s+1}.$$

Comparing this with Mr. Haswell's formula, which is

$$\text{Safe Load} = \frac{8\,W\sqrt{h}}{1.125},$$

we perceive that for the $\frac{1}{2}$-in. penetration, to which the latter applies, it becomes $Q = \frac{2\,W\,h}{1.5}$, and if we assume a fall of 16 ft. we shall have by the Wellington, $Q = \frac{32\,W}{1.5}$; and by Haswell, $Q = \frac{32\,W}{1.125}$, results differing only in the factors of safety. That is to say, with the same actual factor of safety the results for the two formulas would be identical for the assumed fall of 16 ft.

The assumed factor of safety in the Wellington formula is 6, while the assumed factor in the Haswell is only $4\frac{1}{2}$; actually, the above Wellington expression gives $Q = 21.333\ W$, and the Haswell $Q = 28.444\ W$.

* Mosely's Mechanics of Engineering, p. 589.

Mr. Crowell.

The two will be found to give absolutely the same working result with a fall of 28.45 ft. A study of the diagram, Fig. 1, of the characteristic plottings of the respective formulas may be instructive. The Haswell formula is the equation of a parabola and Wellington's is a straight line; the abscissas are the heights of fall of the ram and the vertical ordinates the safe loads expressed in units of ram weight. It

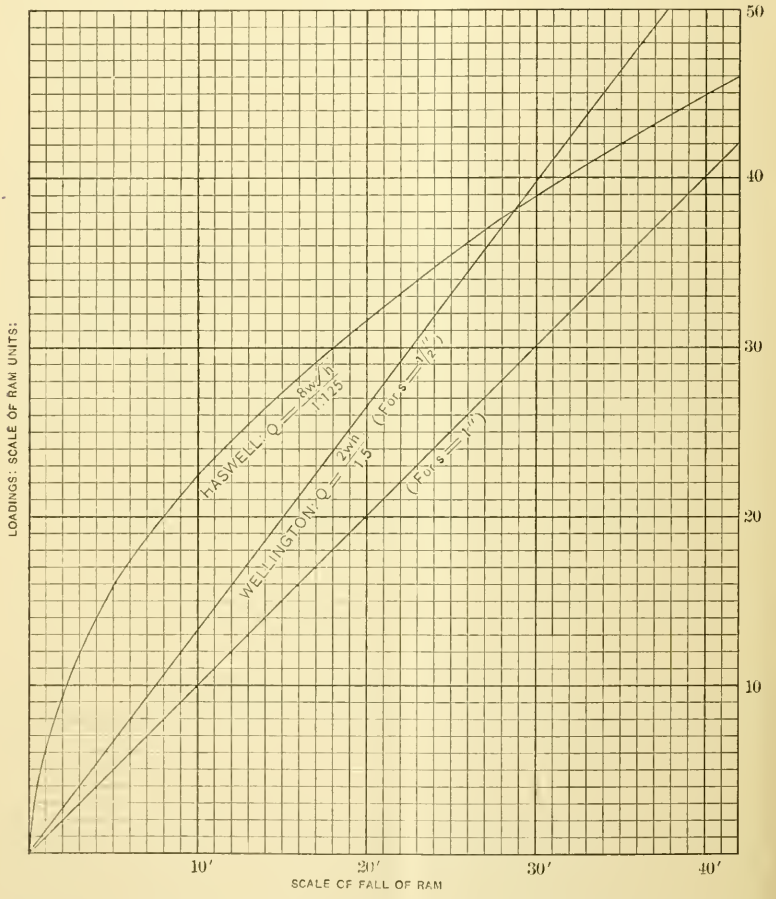

Fig. 1.

will be noted that for low falls the Haswell formula gives relatively great safe loads and for high falls relatively smaller loads.

Considering that the highest value given by the Wellington line has a theoretical factor of safety of 6, it is apparent that there is no advantage in the drop of the Haswell curve at the higher falls. On the

other hand the exceeding great values, relatively, given by it for the low falls show its inconsistency and the danger of its use by the inexperienced. It will be seen that for a fall of only 2 ft. the indicated safe load is one-third as great as that indicated for a fall of 18 ft., nine times as great. Mr. Crowell.

The lower of the two inclined straight lines in the diagram is introduced to show the Wellington values when the penetration is 1 in. It will be observed that the margin of safety is somewhat decreased, so that, strictly, the supplementary quantity in the divisor should not be a constant, but a variable, as was pointed out by the speaker in the discussion on his own paper on this subject, presented before the Society several years ago.* There can be no question, however, that for ordinary penetrations occurring in practice the Wellington formula is amply safe and, in the speaker's opinion, its simplicity and ready applicability entitle it to preference.

E. Sherman Gould, M. Am. Soc. C. E.—It is needless to say that, in the writer's opinion, this paper has great practical value. The long experience of the author in the kind of work described, and his mature judgment, give importance to his utterances. He exposes the vast array of irrelevant elements introduced by many authorities into a problem which, from its nature, does not admit of mathematical refinement, and presents a formula which, in the writer's opinion, has much to recommend it. Its chief peculiarity is in assuming the constant value of $\frac{1}{2}$ in. for the final set, instead of a varying one. That is to say, his formula assumes what amounts practically to "refusal," or the absence of further penetration after the last blow of the hammer. Mr. Gould.

The problem which confronts us in pile-driving formulas, is to find the true relation between the effect of the sudden impact of a falling heavy body, and the steady pressure of a quiescent one. No such relation has yet been discovered, and all that can be done is to seek an empirical expression which will be a safe, practical guide. The formula quoted by the author as "Wellington's," is probably the one which has most endeared itself to American engineers from its simplicity and generally safe results. Its denominator is a binomial expression containing the variable set of the pile, or its final penetration, plus unity. In this form it is relieved of the absurd results which would otherwise ensue from taking very small values for the set, which absurdity would reach its climax if we should assume absolute, or zero set, when the formula would give an infinite bearing capacity to the pile, no matter how light the hammer or how low the fall. This impossible result is avoided by using $s + 1$ in the denominator.

* "Uniform Practice in Pile-Driving," *Transactions*, Am. Soc. C. E., Vol. xxvii, pp. 108, 596.

[Mr. Gould. Wellington's formula, however, like all others recognizing movement or set in the pile under the last blow, fails to meet the requirements of the problem. It is merely the statement of the equality between the work stored in a heavy body falling through a certain height, and a force, called the "safe load" moving through a relatively short distance called the "set." There is no quiescent pressure involved, nor can there be. Work in the hammer can only be offset by work in the pile, and work implies movement and not rest. The "safe load" is not, therefore, according to these formulas, the steady pressure that the pile could bear at the moment of receiving the last blow, but the equivalent pressure which would at that moment force the pile down the distance of the final set.

After receiving the blow it may be assumed, on empirical principles, that the pile could bear that weight without any further movement.

These considerations appear to the writer to meet the question: What formula can be used safely in the case of a set of considerable amount, 6 ins., for instance? The answer appears to him to be: None at all. He considers that, in order to use any formula successfully, the piles, in all cases, should be driven to a final set of not more than 1 in. If a pile 40 or 50 ft. long continues to go down as much as 6 ins. under the hammer, then, to apply a formula, the weight or fall should be reduced until only 1 in. set is produced, and its bearing capacity then calculated by whatever formula may be preferred. If the final set is taken at 1 in., then Wellington's formula reduces to Wh.

In regard to the Wellington formula, which the writer has always been inclined to favor in the above modified form, it must be understood distinctly that, like all empirical expressions, its use applies only to a very restricted range of variation, such as is ordinarily met in practical pile-driving. That is to say, it is not to be taken as a general proposition in applied mechanics. If we so regard it, we shall be involved in the practical absurdity of admitting the effect upon the pile to be the same whether a 50-lb. hammer and 1 000 ft. fall were used, or a 5 000 000-lb. hammer and 0.01 ft. fall.

If specifications called for a hammer weighing 2 000 lbs. and a fall of 20 ft., giving 40 000 foot-pounds, and it were found that only a 1 500-lb. hammer were available, it might, under the circumstances, be admissible to use the lighter weight with a 27-ft. drop, but not one of 1 000 lbs. and a 40-ft. drop, the effect of which would be quite different from that intended.

Mr. Haswell uses the square root of the height, instead of the height itself, in his formula. The writer is inclined to believe that this is correct, as regards the work practically done by the hammer, and with the distinct understanding that the formula applies, like the

Wellington, only to comparatively small differences of fall. The formula also contains a factor of safety, varying from 3 to 6. Taking the higher value, and using round numbers, it will be seen that Mr. Haswell's formula reduces to Mr. Gould.

$$L = 5\ W \sqrt{h},$$

in which L = the safe ultimate load, expressed in the same unit as the weight of the hammer.

It has already been shown that Wellington's formula, for inch refusal, reduces to

$$L = W h.$$

Within the ordinary range of height of fall and admitting a refusal of 1 in. in both, the two will not differ very widely, and the writer is inclined to believe that Mr. Haswell is correct in minimizing the effect of height, particularly as this tends to encourage the use of heavy hammers and low drops.

The whole subject of pile-driving is one of judgment rather than of mathematics. Mr. Le Conte has mentioned some of the curious changes in the set of a pile brought about by time, according to the character of the ground penetrated. It seems, however, hardly worth while to consider the case of such materials as sand, gravel or hard-pan. Generally, when such hard formations are reached, pile-driving is stopped and a case of bearing piles is presented. Soft material under or above water is what we are most concerned with, and it is a matter of common observation that piles driven in such material become so bound by skin friction, that, after 24 hours of repose, their refusal becomes much greater than during the original driving. This seems to be true to a marked degree in the case of jetted piles. In such material piles should be sent down as rapidly as possible by a continuous volley of quick blows. This would work havoc with our formulas, unless the pile driver were sent back again next day to test the piles for final set; but the practical result would be superior.

Horace J. Howe, M. Am. Soc. C. E.—The results of Mr. McIlwain's experiments in Brooklyn have been published in many papers. One of his conclusions is, that the effect of increasing the fall of the ram is to increase the sustaining power of the pile in the ratio of the square root of the fall. Mr. Howe.

Another conclusion was, that by adding to the weight of the ram the sustaining power of the pile was increased by $\frac{7}{10}$ to $\frac{9}{10}$ of the amount used to the ratio of the augmented weight of the ram.

As a result of Mr. Haswell's experiments with 1-lb. weights, he presents a formula for the safe bearing power of a pile, in terms of the weight and fall of the hammer, an assumed penetration of $\frac{1}{2}$ in. or less (which does not appear separately) and a factor of safety. As in other instances, this safe load is found varying directly with W, and a

Mr. Howe. function of h and inversely as F; and there seems to be no new term added to make the formula distinctive.

Passing from the question, as to whether the experiments have been interpreted properly, to the general question of the application of a formula to common practice, there seems to be the necessity of defining, as far as possible, the limit of the reliability of any formula.

All that can be claimed for any formula for the safe load on a pile is governed by the condition of the ground at the time of driving.

If we follow the career of a pile after the formula has left it, we enter into the domain of history, and it is a question of how far into history we care to go. If we wish short-time tests, say a week, or a year, we may arrive at certain conclusions. If we wish long-time tests, say 40 or 50 years, we may arrive at certain other conclusions, or our successors may. But a formula is only a neat expression for a very short-time test indeed, and the emphasis, at the present time, should be placed on the deleterious action of long-time, and more or less continuous, forces.

For instance, take the movement of a railroad train near a building. The speaker has sat in a frame house 1 000 ft. from the track, and has been astonished many a time at the noise made by the structure, due to the vibration transferred through dry loamy soil to the masonry foundations, and thence up to the top story.

Again, another example is a factory on Fort Point Channel, Boston, in which the movements of the piles are seen plainly when the machinery is in motion. A long pendulum is set up inside the building to chronicle the changes. The building is new, and the blue clay is pretty reliable, and, therefore, the settlement, so far, has been slight.

Sometimes we see or hear of a layer of concrete 5 or 6 ft. thick going into a foundation, without, perhaps, being able to find the man who figured the exact thickness required. In the light of these principles, and with a pile foundation, it may not be altogether a waste of money.

Then, water is another long-time agent, tending not only to convey lubricants to the pile, but to move the adjacent soil perhaps laterally.

The matter of incident decay, too, is of the widest interest. It involves a knowledge of the proper elevation above low water at which to cut off the pile or to limit the grillage in different soils and for different purposes. The determination of this limit is still in the experimental stage, and depending on the years as an umpire of opinions.

Then another point, beyond the reach of the formula, is the relative supporting power of one pile, and of a mass of piles; or the relative supporting power of a small and of a large surface. This is also in the experimental stage.

Meanwhile, the pile, with its never-sleeping load, takes advantage of any errors of judgment on the part of the engineer, and slips down and

down and never rights itself. So that, for work which is to be perma- Mr Howe.
nent, the emphasis should be on the long-time experiments.

Ex eriments to the ultimate, of any kind, are rare, and it may be the proper time to say that it is scarcely creditable to the profession. Recently, in a room full of engineers, the question was asked as to ultimate experiments, and there was no answer.

The profession is indebted to Mr. Haswell for again bringing this condition of things to its attention.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer is much Mr. Le Conte.
interested in this subject, and is pleased with the manner in which it is presented. The author is correct in his criticism of existing formulas. The true cause for this extraordinary discrepancy in computed results, as determined by standard formulas, is to be traced to the fundamental fallacy of assuming that the resistance of the soil to the penetration of the pile is constant, for any given depth, during the time the pile is being driven, and that this condition remains constant after the pile is driven. Both of these assumptions are absolutely untenable. Hence the writer has considered the problem as being largely beyond the reach of mathematical formulas which demand rigid facts as a foundation, and this rigidity does not exist. The following notes, based on personal experience, will go to show the absurdity of relying upon the computed results as obtained from the use of the ordinary standard formulas.

Case 1. Sandy Soils.—Soon after driving begins, a cone of resistance forms at and near the foot of the pile, and the material comprising this cone becomes more and more compressed as the pile descends, the penetration at each blow becoming less and less until it reaches a final minimum, of, say, 0.5 in., when the driving is completed. Now, wait 24 hours or more, and, by way of experiment, begin driving a second time. It will be observed that the same pile now penetrates 4 to 5 ins. at the first blow. What does this mean? It simply means that the compressed material surrounding the pile has had time to dissipate, or readjust itself, so that the soil has resumed its normal condition, such as existed before any driving was done. It would appear, therefore, that, in sandy soil, the penetration at the last blow is a very dangerous and misleading element to figure on, for the reason that it is dependent upon a temporary, abnormally compressed condition of the soil, which is always short-lived, and cannot be depended upon as a reliable function in any formula. It is evident that, in sandy soil, the Dutch formula, by *pro rata* of number of blows, is, for obvious reasons, preferable to the final-penetration formulas.

Case 2. Plastic Muds and Clays.—The physical conditions are entirely different from those in Case 1. It is true that the penetration at each successive blow becomes less and less until the driving is completed, that at the last blow being, say, 3 ins. Now, as before, wait 24 hours

Mr. Le Conte. or more and begin driving the same pile a second time. It will be found that the hammer, after repeated blows, cannot start the pile at all. In this case, then, the penetration at the last blow, 3 ins., is no index whatever as to what load the pile will withstand. The blow necessary to start the pile after standing at rest for 24 to 48 hours is the better test. This curious physical property of the plastic muds and clays has been properly likened to the similar results obtained by forcing a pin into a solid india-rubber ball. The same force which pushed it in will pull it out again if it is pulled out immediately; but, after waiting 24 hours, the force required will be five times as great.

Case 3. Hardpan and Sandy Clays.—In this material the most perplexing results of all are to be found. The writer has never seen any two hardpans which developed the same results. The ultimate result depends, not only upon the percentage of clay and sand in the hardpan, but also upon the solubility of the clay when brought into contact with the local ground-water. Where piles are driven through ground-water overlying hardpan, the water invariably follows each pile down as it penetrates the ground, and softens the clay in contact with the surface of the pile, and often practically destroys all lateral friction. In such cases the only point of support is at the end of the pile, and the overhead load will be supported by a cluster of columns (piles), the heights of which are equal to the length of the piles.

Finally, there is no doubt about the fact that the best way to test the bearing power of piles, in any given ground, is to drive experimental piles to the required depth, then load them with pig-iron and observe the effects after long standing, and thus find by trial their ultimate resistance. This is not always practicable, in which case the next best way is to drive experimental piles to the depth required or specified, then let them rest for 48 hours or more and drive them a second time, and note the drop of the hammer necessary to start the pile and the first penetration in inches. The results thus obtained represent more nearly the resistance of the soil in its normal condition.

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PAPERS AND DISCUSSIONS.

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THE GROINED ARCH AS A COVERING FOR RESERVOIRS AND SAND FILTERS: ITS STRENGTH AND VOLUME.

Discussion.*

By Messrs. L. J. Le Conte and William R. Hutton.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—This paper shows much careful study and research, while the tabulated results add very materially to its value. The writer will confine his remarks to the economic features of construction as applied to the covering of reservoirs and filters in general.

The masonry coverings considered by the author are extremely pleasing to the eye and are highly satisfactory in every respect except as to cost. There are many modern designs for masonry coverings, composed of concrete and iron tie-bars combined, which are equally durable, strong and efficient, take up less space inside and are cheaper and better in every way. Taking the author's limiting planes, for volumetric comparison, the modern designs with span of 25 ft. and thickness of only 2 ft. would require about 0.70 cu. ft. of Portland cement concrete per superficial foot of roof covering, and the expense would not exceed 50 cents per foot; if the spandrels were not filled in with masonry, the cost would not be more than 35 cents per superficial

*This discussion (of the paper by Leonard W. Metcalf, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for April, 1899) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to October 20th, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

Mr. Le Conte. foot of roofing. Roughly speaking, 1 000 cu. ft. of good Portland cement concrete are enough for 1 875 sq. ft. of covering. Hence, the writer thinks that, in course of time, financial reasons, more than any others, will drive groined arches from the field of competition.

Mr. Hutton. William R. Hutton, M. Am. Soc. C. E. (by letter).—The author remarks that all the examples of the groined arch in engineering structures that have come to his knowledge are in the United States. These applications are not new in Europe, however. In 1863–65 Mr. Belgrand built the reservoirs of Menilmontant and Belleville in Paris, both of two stories. The lower story is covered with semi-circular groined arches of 16 ft. clear span on piers 4 ft. 8 ins. square. The arches are 14 ins. thick at the crown, and are leveled up with masonry. The upper reservoir is covered with groined arches of 18 to 20 ft. span, and 2 ft. 8 ins. rise; the arches being formed of two thicknesses of tile laid in cement, breaking joint. Their total thickness is less than 3 ins. Upon this, 18 ins. of earth and sod are laid.

Ten or more years later, the reservoir of Montsouris was built in the same way. The lower reservoir is covered with groined arches in rubble masonry 14 ins. thick at the crown; the haunches being filled with concrete or masonry. The upper reservoir is covered by a series of groined arches less than 3 ins. thick, on spans of 18 ft., with a rise of 2 ft. 8 ins. This thin arching is the "Guastavino" method, recently introduced into America by the architects. The writer has not investigated the origin of this system of arching as applied to reservoirs, which has been used by others than Belgrand, and in other places. The "Guastavino" method is much older. The Church of St. Eugene (Paris), in 1854, was ceiled and roofed with Gothic ribs of iron, the arched surfaces between the ribs being of tiles laid flat, in cement.

The upper reaches of the Canal St. Martin, in Paris, rest upon gypsum. The material in places has been dissolved out from under two of the upper locks and from one short level, by the filtration of water from the canal and from the Basin of La Villette. These have been repaired by means of piers sunk 30 ft. and 40 ft. to solid ground, and from them spring groined arches of brick or rubble to form the floor.

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THE ARTIFICIAL PRESERVATION OF RAILROAD TIES BY THE USE OF ZINC CHLORIDE.

Discussion.*

By Mr. SAMUEL M. ROWE.

SAMUEL M. ROWE, M. Am. Soc. C. E. (by letter).—These researches by Mr. Curtis certainly have much value, not, perhaps, for any definite conclusions adduced, or that can be adduced, so much as for giving a *résumé* of the question of timber preserving from the time of the report of the Committee of this Society in 1885 to the present time. That this article is resulting in an awakening to the importance of this matter seems to justify its preparation and the great amount of labor and care required, and it is to be hoped that something more than thanks will be accorded. Mr. Rowe.

The pressing needs of the day for cross-ties to supply the waste on existing railroads and for new lines has called attention sharply to the necessity of some method of preventing the rapid depletion of the available stock of timber, particularly as few American railroads are yet prepared to resort to metal ties.

The discussion of this matter, both in this case and generally, is somewhat confusing, but probably pertinent and worthy of consideration. To extricate ourselves from this, however, and to place ourselves on something like solid ground, we are inclined to fall back on experience and give some of the conclusions derived therefrom.

It came into the province of the writer in 1885, to be placed in charge of the works then just completed at Las Vegas, N. Mex., a

* Continued from August, 1899, *Proceedings*. See April, 1899, *Proceedings* for Paper by W. W. Curtis, M. Am. Soc. C. E., on the subject.

Mr. Rowe. position in which it was imperative to make a close study of the philosophy, the practical workings, the methods used, the nature of the chemicals, as well as a thorough analysis of the results attained. The zinc-tannin or Wellhouse patent process was to be used, and in this study the aid of O. Chanute, M. Am. Soc. C. E., and the late Joseph P. Card, Assoc. Am. Soc. C. E., was availed of; they being owners of the then existing patents on the process. It is unnecessary to say that the conclusions there reached were and are still of much value, no occasion for revision to any extent having arisen.

It is a pleasure to read the contribution of Mr. Noyes, in reference to these works, and it is very desirable that more such men enter the field to add to the number of careful observers.

It is not deemed necessary or desirable to go deeply into this discussion at the present time, and the writer will confine his remarks to a general survey of the practical points of the situation and to the importance of educating practical operators, of whom there are too few.

We should not place particular stress on the process used, whether it be the Burnett, the zinc-tannin or the creosote process, the appliances and methods being quite similar, but on correct and efficacious methods, the main point being to get the absorption of a proper amount of chemicals, or rather to prepare the timber for it. To do this, a thorough knowledge of the absorptive qualities of the timber to be treated, as to density, dryness, etc., must be had, after which judgment and discretion, with some degree of practical experience, will be the best guides.

In regard to the question of securing any desired degree of absorption, it depends on the density of the timber; but as to the effectiveness of the process used (steaming or the vacuum), there is little question. By natural absorption, that is, by soaking or immersing in water, would require, under the most favorable circumstances, some thirty days to absorb as much as will be injected by this method in a few hours (five or six altogether).

It should be remembered, too, that the amount of chemicals, or antiseptics, has not all to do with the results, neither can a superabundance of the chemical be relied upon to make amends for lack of judgment in respect to other parts of the treatment. The overplus above that which long experience and trial in the track has shown to be necessary is worse than money thrown away. The saying in the "Hoosier Schoolmaster," "While yer gittin git a plenty," will hardly hold in this case.

The question of choosing open-grained timber is to some extent misleading, as the writer's observations for the last 15 years lead him to believe that the expression "the better the timber, the better the tie," is in the main true, the fact remaining that many of the timbers

now deemed worthless for this purpose, may be made lasting and serviceable. Mr. Rowe.

The mention of small ties, 6 x 8 ins., deserves notice. A sawed tie less than 7 by 9 ins. can on no rational grounds be defended.

The degree of drying which a tie should have should be sufficient to relieve the wood of at least half its water; the time required to do this depending on climate, moisture and other conditions which vary in different parts of the country. There may be exceptions as to this with timbers which have not yet come under the writer's notice.

If the writer were permitted, he would say to the railroad manager, that in many cases such a plant, equipped and operated, would prove of as much advantage as a repair shop, and should have the same attention and care; the manager of the works being educated in this particular line and equally as well paid as the master-mechanic of his repair shop. No man can be prepared for this without close contact with the practical part of the work and a close study of its operation.

As an example of this, one of the largest railroad systems in the United States, put aside all theories and speculations as to particular methods and took the tangible results shown on the financial sheets of cost of maintenance, viz., a reduction of renewals from over 20% in 1885, to less than 5% in 1897-98.

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ON THE THEORY OF CONCRETE.

Discussion.*

By Mr. GEORGE W. RAFTER.

Mr. Rafter. GEORGE W. RAFTER, M. Am. Soc. C. E.—The author desires to express satisfaction at the appreciative discussion of the paper. That there is an important place in the literature of concrete for a paper of this character is well indicated by the remark of Mr. Fowler:

"Many writers on this subject have assumed that every concrete, no matter what the proportions may be, has simply a yard of broken stone, without regard to the fact that there is more or less mortar in different proportions."

The author has been greatly surprised, in looking over the literature of concrete, to see how far this fallacy has been accepted. It has also been matter of surprise to find a nearly absolute dearth of definite information as to the amount of mortar and concrete produced by definite proportions of cement, sand and broken stone, as well as the unit weight of the resultant concrete.

This paper, taken in connection with the more extended information given in the report to the State Engineer and Surveyor of New York, is an attempt to supply, in some slight degree, what appears, after study of the question, to be a very material deficiency. The results are, however, unsatisfactory to the author in this particular; that, having laid out the plan of work, no special study of the data was made until about two years after the completion of the tests. This was a mistake. The data should have been tabulated and studied

* Continued from August, 1899, *Proceedings*. See April, 1899, *Proceedings*, for Paper by George W. Rafter, M. Am. Soc. C. E., on the subject.

from day to day during the progress of the tests. Had this been done, there seems to be no doubt that some of the irregularities of the results could have been eliminated. Mr. Rafter.

The subject is extremely complicated, many apparently insignificant elements entering in to detract from the strict, scientific accuracy of the results. The more especially is this true, as in the present case, where an attempt has been made to keep the results on practical lines purely, using only such methods as could be applied in practice on work, without adding unduly to the cost of concrete. However, taking this latter condition into account, the author reiterates the view expressed in the paper, that the results are very good.

Occasion is taken at this place to especially call to the attention of those participating in the discussion, that the results of these tests are given in great detail in the annual report of the State Engineer and Surveyor of New York for 1897.* The results of a previous set of tests made in 1893 are given in the annual report of the State Engineer for 1894. Reference to all these reports should be made by anyone wishing full data.

As regards mixing concrete, either by hand or machinery, the author's experience is that with proper supervision, good concrete can be obtained by either method. As remarked by Mr. Tillson, the difficulty in hand mixing is to insure thorough work. On this point the author has found that by drilling a gang of laborers to work strictly according to formula, performing each operation in the same way, every time, there is little difficulty about securing fairly uniform results by hand mixing. Nevertheless, the author's opinion is that proper machine mixing is preferable, as leading to a uniformity of product which, while possible to attain by hand mixing, is still somewhat difficult. In a general way, the author agrees, therefore, with the statement of Mr. Gould, that no hand-made concrete, using the same proportions, can equal a machine-made product, because of the thorough way in which the machine churns up the entire mass.

The tests show that dry concrete is considerably stronger than that made with excess of water. As stated in the report to the State Engineer and Surveyor, the author's experience has taught him that the only way in which dry concrete can be safely used, especially in warm weather, is to wet the surface thoroughly as soon as the ramming has been completed, and to keep the surface wet until after the completion of the setting. This procedure will be likely to prevent the formation of the partings between successive beds referred to by Mr. Gould. The author, however, does not intend to be insistent as to the use of dry concrete, but rather presents the results of these tests for comment and professional criticism. He is, nevertheless, well satisfied that dry concrete, with the mortar comprising 40% of the aggregate material,

*See also "Report on Tests of Metals and other Materials," Watertown Arsenal, for 1898, where may be found, also, results of other concrete tests.

Mr. Rafter. can be made practically water-tight. In recent work with a concrete of this composition, a dam has been constructed, which, with one exception, shows absolutely no moisture on the face. The one exception is two pin holes, both within an area of 1 sq. ft. Their presence can only be taken to indicate careless work—for the moment, evidently, vigilance was relaxed—but this single exception can hardly be taken as proving the inapplicability of dry concrete. In any case, it is certain that plastic and excess concretes can be used much more easily than dry, and the practical question is, after all, whether the gain in strength is enough to compensate for the increased chance of poor work.

As to ultimate strength, it seems certain that time is a great eliminator of the differences between dry, plastic and excess concretes, the tendency being undoubtedly to show less and less difference in ultimate strength with increase of age.

The concrete blocks here discussed were made between July 10th and September 5th, 1896. They were broken between January 29th and May 10th, 1898. The ages of the blocks, therefore, varied from about 670 to about 570 days. The exact age of each block may be obtained by reference to Table No. 1 of the report to the State Engineer and Surveyor, in the annual report for 1897.

One important point which the author had in mind in making these tests was to show that, for many purposes, the formulas in common use give concretes which are unnecessarily expensive. Thus, for foundations which are entirely under water there is no object, so far as the author can see, in spending the money necessary to make an absolutely impervious concrete. For such work a 30 or 33% concrete is apparently sufficient. The same is also true of retaining walls, for which concrete is now used extensively, and through which weep holes can be left, thus insuring that water will not penetrate any considerable distance into a 30% concrete, and which may, as the author is well aware, contain a slight percentage of voids.

Where water-tight work is required, the mortar may be made from 40 to 45% of the aggregate. By using different sized stone for the aggregate, there seems to be no doubt that 40 to 42% of mortar will make a practically water-tight material. The author says practically water-tight, because the quality of the mortar will, in some degree, decide as to absolute water-tightness.

In the original tests of 1893, the ratio of broken stone to mortar was made mostly 42%, a few blocks having been fabricated with the ratio of 30 per cent. These blocks were all broken under the author's personal supervision, and none of the 42% blocks exhibited any voids visible to the eye. They were, in the best sense of the word, solid blocks. Blocks of 30 and 33%, however, exhibit some voids, the greatest difficulty with such concretes being to make them absolutely

fill out all the voids next to the forms. This is a difficulty which is, however, very easily corrected by using a somewhat larger proportion of mortar next to the forms, as has frequently been done with good success on work. Mr. Rafter.

In the series of tests made in 1896, and which are here specially considered, the author is unable to say as to whether there were any appreciable voids in the 33% concretes, but from his experience with the 30% concretes made in 1893, he would be disposed to say that the voids were very small, especially in the plastic and excess concretes. Under this head it may be pointed out that the effect of ramming is to decrease the voids. This was strongly brought out by a series of tests in which it was found that the Portage stone, broken to pass through a 2-in. ring, when slightly shaken in the measure, gave, as a mean of five trials, 43.3% voids. The same stone packed in a measure with a tamping iron, used about as forcibly as the ordinary ramming of concrete, gave, as a mean of five trials, 37.4% voids. In these tests the air was driven out of the voids entirely.

The author has little doubt that, with the lubrication of the mortar, the voids are reduced, under thorough ramming, to not much more than about 30 per cent. It is from this point of view that he decided to use 33% of mortar in the tests herein discussed, that quantity of mortar being enough to just about insure filling the interspaces of the stone. Such a concrete will not, however, be absolutely water-tight.

In the same way the excess of mortar in 40% concrete was considered to be large enough to insure practical water-tightness. It is possible, however, that slightly larger proportions than the foregoing should, in some cases, be used. On the other hand, by selecting aggregate materials of different sizes, thus reducing the proportion of voids, it is believed that the proportions of 33 and 40% fairly meet the great majority of cases.

Mr. Conrow points out that the average ratios of strength are, for concretes with dry mortars, 29.1; plastic, 26.6; excess, 25.3; as based on the average of ten tests. From these figures he deduces that dry concrete is about 15% stronger than wet. In his experiments as to relative cost of laying wet concrete versus dry concrete, he found that wet concrete could be fabricated at a cost for labor of $1.13 a yard, dry concrete costing, with other conditions the same, $2.12 a yard. This is for hand labor. The author's experience is that for machine-mixed concrete, aside from ramming, the cost will be no greater for dry than for wet. As an average, the ramming of dry concrete will cost 25 cents per yard more than for wet.

Mr. Hazen's discussion of the conditions involved in the determination of voids, and as to the weight of concrete, illustrates forcibly how far we are from appreciating the practical considerations to be taken into account in making concrete, especially when the subject is

Mr. Rafter. approached from the theoretical point of view purely. However, as to determining the voids, the use of the method outlined by Mr. Hazen seems to the author to give results too large. Independent of its lack of applicability out on work, the author believes that the displacement method of determining voids is for clean material, much the better. A knowledge as to the interstices in the aggregate material or ballast is wanted and not, in any degree, the porosity of the material. The author does not wish to be understood, however, as saying that information as to the porosity of aggregate material may not be of importance, the more especially since such information is very necessary in order to decide between different samples. His point is, that, as regards concrete mixtures, what is wanted is the volume of the interspaces between the stones, independent of the porosity. Limestones, sandstones and other aggregates are often quite porous, and the method by specific gravities apparently takes into account this porosity. Limestones sometimes absorb 5% of their weight of water, and sandstones as much as 7%, while broken brick may absorb 20 per cent. As fair averages, we may take limestone at 3%, sandstone 4%, and broken brick 10 per cent. The most practical way to proceed is to first thoroughly wet the broken stone and allow it to drain before placing it in the measure. Water should then be poured in slowly and at one corner of the measure, thereby insuring the driving out of the air. For sand, absorption into the pores is so small that it may be neglected, but it is important to pour in the water slowly and at one corner of the measuring box. This procedure applies to clean sand. For sand containing much vegetable mould or other foreign matter, Mr. Hazen's remarks as to the uncertainty of the displacement method apply forcibly. As regards the penetration of water into the pores of the aggregate material, it may be remembered that the smearing of the broken stone with mortar tends to seal up the pores in properly-made concrete, thus preventing the ingress of any considerable amount of water. In any case, taking into account the porosity of the aggregate material is a misleading refinement.

There is no reason why, with care, any ordinarily intelligent foreman cannot, by the foregoing simple method, determine quickly the voids in aggregate material and clean sand, as often as required, and well within any limit of accuracy needed for proportioning concrete.

Reference has already been made to the original series of tests of 1893, and which are reported upon in the annual report of the State Engineer and Surveyor for 1894. The weights of the 42% blocks fabricated at that time ranged from about 145 lbs. per cubic foot to about 150 lbs., the variation in weight being in some proportion to the strength of the mortar, blocks with 1 to 1 mortar being heavier than those with 1 to 2 or 1 to 3. In some cases, cubes of 1 ft., with 1 to 1 mortar, 42%, weighed from 150 to 160 lbs. The tests of the series of 1893 are,

Mr. Rafter.

however, unsatisfactory in this, that the exact dimensions of the blocks were not taken, and there is probably some uncertainty in the figures on this account.

In the tests of the series of 1896, and which are here specially considered, no such uncertainty exists; the exact dimensions of each block were taken carefully and the weights per cubic foot computed therefrom. In Table No. 15 we have the mean weights of all the concrete blocks in pounds. The Portage sandstone used as ballast weighs 155 lbs. per cubic foot, and has, when crushed in 12-in. cubes, a crushing strength of 6 000 to 7 000 lbs. per square inch. A few determinations of weight of mortar used show the following mean weights per cubic foot:

			Weight per cubic foot.
Empire	cement—	1 to 2 dry	126.4 lbs.
"	"	1 " 3 "	120.9 "
"	"	1 " 4 "	119.1 "
"	"	1 " 2 plastic	124.7 "
"	"	1 " 3 "	121.0 "
"	"	1 " 4 "	119.2 "
Iron Clad	"	1 " 2 dry	128.5 "
"	"	1 " 3 "	124.8 "
"	"	1 " 4 "	120.9 "
Champion	"	1 " 1 "	126.2 "
"	"	1 " 2 "	123.8 "
"	"	1 " 3 "	119.2 "

The foregoing weights of mortar blocks, as well as concretes, included in Table No. 15, are all as taken at the time of breaking, and when, presumably, the excess water had evaporated. This conclusion is reached by considering that the blocks stood in a dry, warm place for some time before breaking.

Referring to Table No. 15, it appears that there was very little difference between the weights of 33 and 40% blocks. With the Genesee cement 1 to 1 mortars, the mean weights are, for 33% blocks, 146.5, and for 40% blocks, 145.6. For Wayland, 1 to 1 mortars, the mean figures are 33%, 146.9, and 40%, 145.8. For Genesee 1 to 2, the figures are, 143.8 and 143.7; and for Wayland 1 to 2, 144.5 and 144.2.

For the Genesee cement, the weight per cubic foot was taken at 99.5 lbs.; for the Wayland, 100 lbs.; for the Iron Clad, 97 lbs.; for the Empire, 99.5 lbs., and for the Champion, 87 lbs. The reason that the weights of 33 and 40% blocks are practically the same is probably because in 33% blocks the excess reduction of voids under ramming just about compensates for the extra quantity of mortar in the 40% blocks, the relation between the weights of mortar and broken stone being such as to give this result. The persistency of the figures through the whole series shows that this result is not accidental, but rather the expression of a well-defined law.

Mr. Rafter.

TABLE No. 15.—Mean Weights of Concrete Blocks.

(In pounds.)

		1 to 1.		1 to 2.		1 to 3.		1 to 4.		1 to 5.		1 to 6.	
		33%	40%	33%	40%	33%	40%	33%	40%	33%	40%	33%	40%
Genesee.	Dry	146.6	.....	.....	143.4	142.7	142.9	141.3	141.6	140.5	.....	139.5	.....
	Plastic	146.2	.....	143.2	143.4	143.2	142.4	142.1	140.7	141.2	.....	.....	.....
	Excess	146.6	145.6	144.5	144.4	142.9	141.9	143.1	141.2	.....	.....	.....	.....
	Mean	146.5	145.6	143.8	143.7	142.9	142.4	142.2	141.2	140.8	.....	139.5	.....
Wayland.	Dry	147.0	146.4	144.5	144.4	140.0	142.1	142.6	140.0	141.7	139.7	.....	.....
	Plastic	147.8	146.4	145.1	143.5	144.1	141.0	141.2	141.6	140.8	139.0	.....	.....
	Excess	145.9	144.7	144.0	144.6	140.7	141.6	141.3	142.2	139.8	140.4	.....	.....
	Mean	146.9	145.8	144.5	144.2	141.6	141.6	141.7	141.3	140.8	139.7	.....	.....
Iron Clad.	Dry	.....	.....	143.7	144.2	141.5	141.6	141.7	140.6	.....	.....	.....	.....
	Plastic	.....	.....	143.0	143.6	141.0	141.1	140.4	140.6	.....	.....	.....	.....
	Excess	.....	.....	143.3	144.2	140.9	141.8	141.9	140.2	.....	.....	.....	.....
	Mean	.....	.....	143.3	144.0	141.1	141.5	141.3	140.5	.....	.....	.....	.....
Empire.	Dry	.....	.....	142.6	144.3	140.5	140.8	139.8	139.7	.....	.....	.....	.....
	Plastic	.....	.....	142.9	144.0	141.5	141.1	141.4	140.4	.....	.....	.....	.....
	Excess	.....	.....	144.6	143.7	141.7	141.7	141.1	139.4	.....	.....	.....	.....
	Mean	.....	.....	143.4	144.0	141.2	141.2	140.8	139.8	.....	.....	.....	.....
Champion.	Dry	142.5	143.1	142.7	140.3	138.9	139.4	.....	.....	.....	.....	.....	.....
	Plastic	142.4	142.8	142.3	141.3	140.6	138.3	.....	.....	.....	.....	.....	.....
	Excess	141.5	142.2	140.5	142.7	139.4	139.0	.....	.....	.....	.....	.....	.....
	Mean	142.1	142.7	141.8	141.4	139.6	138.9	.....	.....	.....	.....	.....	.....

While many of the points brought out by the discussion are of great practical interest, the author wishes to say that many more interesting points could have been touched upon; as, for instance, the change of volume of mortar material when water is added, the effect of ramming on consolidation of mortar itself, the consolidation or reduction of void space in the aggregate material under ramming, the probable greater reduction of such space with rich mortars which act more thoroughly as lubricants, and many others which could be mentioned. The paper itself was purposely left a bare recital of main facts in regard to the tests, of which the detail is given in the report to the State Engineer and Surveyor, the author's hope being that all the various points of view would be taken up in the discussion. That many of them are left untouched, seems to show, on the whole, a lack of appreciation of the theory of concrete; at the same time the author agrees thoroughly with Mr. Hall that there is no reason why the old nomenclature should obscure, from those who are looking for it, the true theory of concrete mixtures. Nevertheless, the fact remains that apparently either that or some other cause has so far obscured the true theory of concrete mixtures that American engineering literature is, in the last year of the nineteenth century, practically barren of rational information as to the theory of concrete.

Professor Baker's view, that 40% concretes should show an excess of strength over 33% concretes in the same proportion to the excess percentage of mortar, is not supported by the results of these tests. The reason probably is that in 33% concretes the strength is largely dependent upon the close bearing of the broken stone, while in the 40% concretes it is, in a somewhat larger degree, dependent upon the strength of the mortar. Professor Baker, however, is right in his conclusion that, as regards final results, the method of storage is not very important for concrete which has, as in the present case, attained an age of at least a year and a half. Had the blocks been broken at an earlier period, they would probably have shown greater differences than those indicated by Table No. 7. Mr. Rafter.

As to the variation in the strength of the concrete, under the same conditions for the different cements, the author has no explanation to offer other than that it is possibly due to varying qualities in the cements themselves which have not thus far been closely differentiated by the methods of tests actually in use.

Mr. Le Conte has indicated the true phase of concrete practice at the present time by his remark that it is a humiliating spectacle to see first-class material manipulated in an irrational and wasteful manner, developing results which cannot be shown to possess a single redeeming feature. Especially is this true as regards the cleanness of the aggregate material. The broken stone should, in every case, be washed clean, and indeed cleanness should be made an indispensable condition at every stage of the operation. That such conditions are not generally insisted upon, merely illustrates how far many of us are from understanding the true theory of concrete. These remarks apply to a considerable proportion of the concrete used at the present time in the United States.

The tests recorded by Mr. Von Schon are very interesting, especially as to the general programme which he outlines. His first condition, to operate practically, avoiding the introduction of refinements which cannot be followed in actual construction, is thoroughly in line with the author's views, as is also the second. The author cannot but think, however, that Mr. Von Schon's tests would have been more satisfactory had they been on the line of crushing rather than breaking bars by applying a load at the center, though it is recognized that this is, after all, merely a difference of opinion; the author's contention being, however, that had Mr. Von Schon broken a number of cubes, the results would have been comparable with most of the results thus far obtained, whereas, as the matter stands, it is impossible to compare them.

By way of showing the variations in the quantity of green mortar produced from the same quantity of cement and sand, but with such variation in the quantity of water as is necessary to produce dry, plastic or excess mortars, the following figures are cited.

Mr. Rafter. For 1 to 3 mortars, $\frac{2}{3}$ cu. ft. of cement and 2 cu. ft. of sand produced quantities of mortar in cubic feet, as follows:

	Genesee.	Wayland.	Iron Clad.	Empire.	Champion.	Means.
Dry.....	2.05	2.12	2.19	2.16	2.13	2.13
Plastic.··	2.21	2.16	2.23	2.25	2.18	2.21
Excess...	2.25	2.16	2.28	2.29	2.15	2.23

For 1 to 2 mortars, 1 cu. ft. of cement and 2 cu. ft. of sand, produced quantities of mortar in cubic feet, as follows:

	Genesee.	Wayland.	Iron Clad.	Empire.	Champion.	Means.
Dry.....	2.51	2.37	2.57	2.50	2.22	2.43
Plastic...	2.58	2.49	2.62	2.58	2.43	2.54
Excess...	2.75*	2.58*	2.62*	2.61	2.48	2.61

Inasmuch as the cement was weighed, thus insuring the same quantity in each series, the irregularities in these results are chargeable to variations in sand and quantity of water.

The author wishes to thank those who have taken part in the discussion for the many valuable suggestions made. In view of the large number of works, now either in progress or contemplated, in which concrete is either in use or to be used, the author hopes that the general question of the rational relation between the voids in the aggregate material and the amount of mortar, especially in relation to unit costs, and other important questions, may be so thoroughly taken up as to lead to final, definite conclusions. The present paper was intended to be largely suggestive.

* One determination only.

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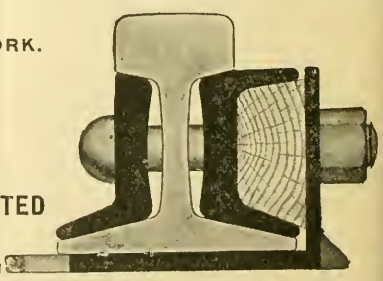

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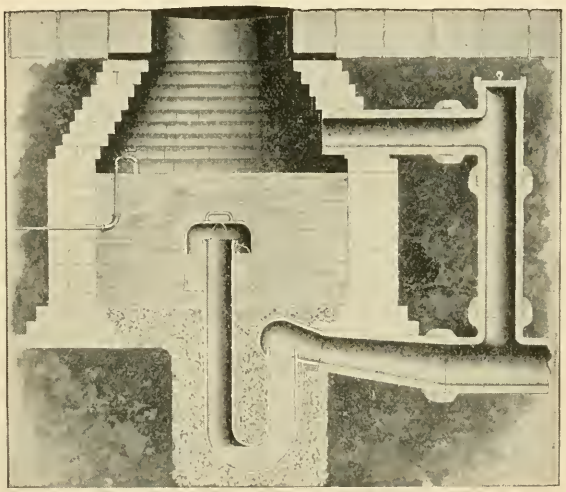

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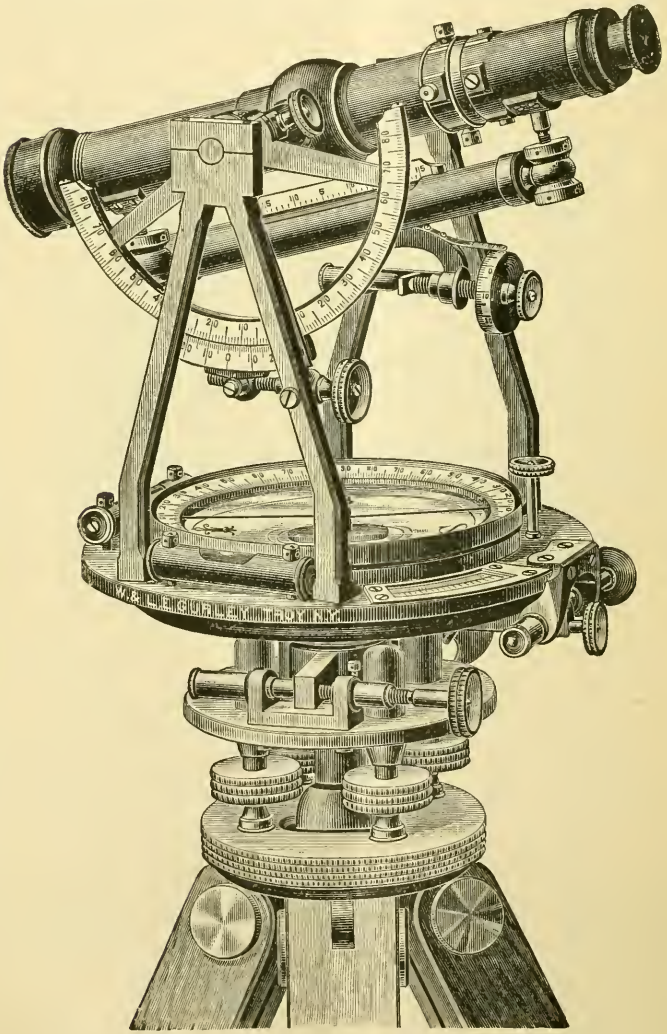

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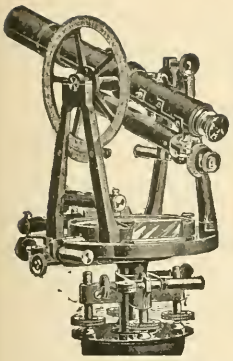

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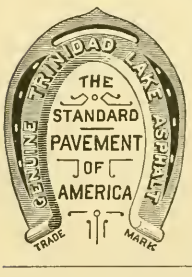

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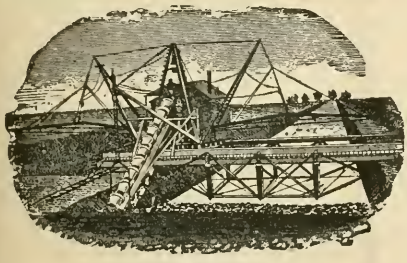

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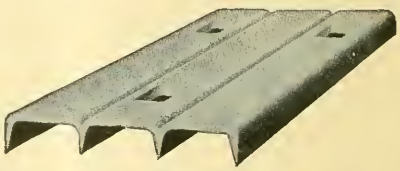

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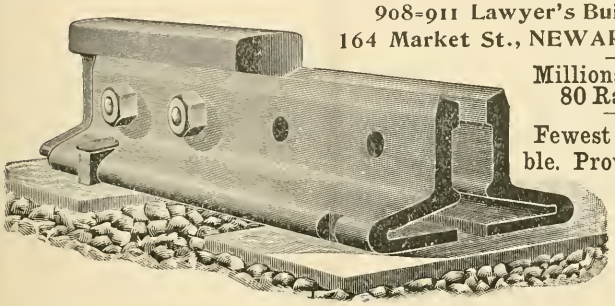

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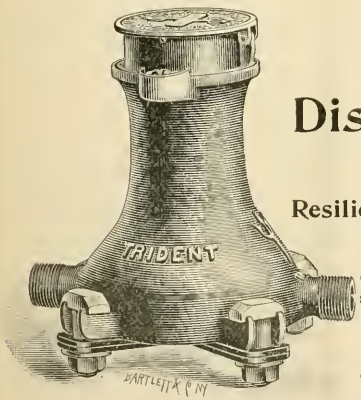

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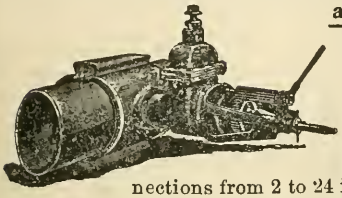

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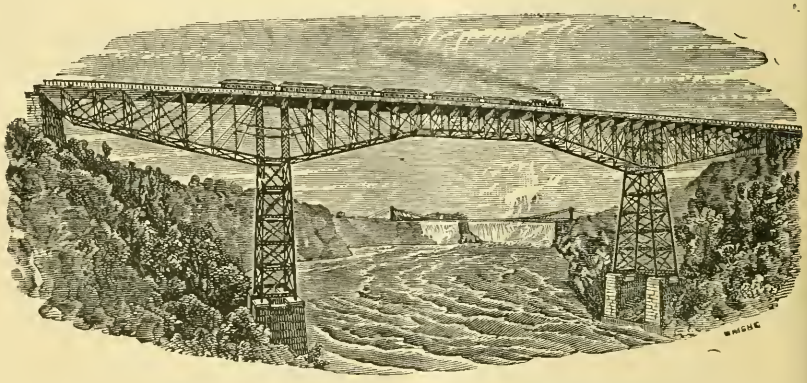

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AMERICAN SOCIETY

OF

CIVIL ENGINEERS

October, 1899

PROCEEDINGS = VOL. XXV—No. 8

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

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Vol. XXV. OCTOBER, 1899. No. 8.

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publications.

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The prices of publications are as follows: *Proceedings*, $6 per annum; *Transactions*, $10 per annum. Postage will be added when *Proceedings* are sent to foreign countries

American Society of Civil Engineers.

OFFICERS FOR 1899.

President, DESMOND FITZGERALD.

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Secretary, CHARLES WARREN HUNT.

Treasurer, JOHN THOMSON.

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Term expires January, 1900:	*Term expires January, 1901:*	*Term expires January, 1902:*
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Standing Committees.

THE PRESIDENT OF THE SOCIETY IS *ex-officio* MEMBER OF ALL COMMITTEES.

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Special Committees.

ON UNIFORM STANDARD TIME:—Sandford Fleming, Charles Paine, Theodore N. Ely.

ON ANALYSIS OF IRON AND STEEL:—Sub-Committee of the American Society of Civil Engineers (of the International Committee on Standards for the Analysis of Iron and Steel, of which Prof. J. W. Langley is Chairman)—Charles B. Dudley, William Metcalf, Thomas Rodd.

ON UNITS OF MEASUREMENT:—George M. Bond, William M. Black, R. E. McMath, Charles B. Dudley, Alexander C. Humphreys.

ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Noble, George S. Webster, O. M. Carter, W. B. W. Howe, Louis C. Sabin, H. W. York.

The House of the Society is open every day, except Sunday, from 9 A.M. to 10 P.M.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER, - - - 533 Columbus.
CABLE ADDRESS, - - - - "Ceas, New York."

Vol. XXV. OCTOBER, 1899. No. 8.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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MINUTES OF MEETINGS.

OF THE SOCIETY.

October 4th, 1899.—The meeting was called to order at 8.40 P. M., Vice-President Edward P. North in the chair; Charles Warren Hunt, Secretary; and present, also, 55 members and 4 visitors.

The minutes of the meetings of September 6th and 20th were approved as printed in *Proceedings* for September, 1899.

A paper entitled "Impact Tests of Structural Steel," by S. Bent Russell, M. Am. Soc. C. E., was presented by the Secretary, who also presented a written discussion on the subject by Mr. C. M. Broomall.

Ballots were canvassed, and the following candidates declared elected:

As Members.

THOMAS HOWARD BARNES, Medford, Mass.
MARTIN JOSEPH CAPLES, Roanoke, Va.
ARTHUR POWELL DAVIS, Washington, D. C.

Chandler Davis, New York City.
Cornelius Donovan, Port Eads, La.
Julian Griggs, Columbus, Ohio.
James Nisbet Hazlehurst, Tampa, Fla.
John Donald Maclennan, Cleveland, Ohio.
William Curtis Merryman, New Martinsville, W. Va.
Alfred Mitton Mosscrop, Rochester, N. Y.
Frederick Palmer, Dehri Bridge, Bengal, India.
William David Pence, La Fayette, Ind.
Naoji Shiraishi, Hamaishiki, Yokkaichi, Miye, Japan.
Horace Augustus Sumner, Alamogordo, N. Mex.
Joseph Till, Freeburg, Minn.
John Lane Van Ornum, St. Louis, Mo.
John Waterhouse, New York City.
Charles Edward Webster, South Bethlehem, Pa.
Morris Alexander Zook, Princeton, Ind.

As Associate Members.

David Guy Anderson, Philadelphia, Pa.
Ernest Edmund Brydone-Jack, Pittsburg, Pa.
Leonard Martin Cox, New York City.
James Hervey Dingle, Charleston, S. C.
John Lyle Harrington, Elmira, N. Y.
Frederick Thomas Llewellyn, New Orleans, La.
Paul Albert Seurot, Ozone Park, N. Y.
Louis Henry Shoemaker, Paterson, N. J.
Walter Tuttle Spencer, Guilford, Conn.
George Draper Stratton, Marysville, Cal.
Robert Andrew Thompson, Austin, Tex.
Taro Tsuji, Tokyo, Japan.

Announcement was made that the following candidates were elected by the Board of Direction, October 3d, 1899.

as associate.

Jean Prosper Auguste Maignen, Philadelphia, Pa.

As Juniors.

Wager Fisher, Axton, N. Y.
Victor Witmer Kline, Detroit, Mich.
Hugh McLeod, Butte, Mont.
John Francis Richardson, New Orleans, La.
Alexander Jenifer Taylor, Wilmington, Del.

The Secretary read the list of nominees* presented by the Nominating Committee for the offices to be filled at the next annual election.

The Secretary read a letter from the Secretary of the Technical Club, of Chicago, Ill., extending the courtesies of the club to members of the Society who may visit Chicago during the Festival Celebration, October 9th to 14th, 1899.

The Secretary announced that the following resolutions were adopted by the Board of Direction, October 3d, 1899:

"*Whereas*, The Society in Annual Convention has requested the Board of Direction 'to take up the matter of Standard Time and settle it,' and with the understanding that this request particularly refers to the use of the 24-hour system of notation as employed in the published matter and correspondence of the Society; therefore, be it

"*Resolved*, That the use of the 24-hour system of time notation in any of the publications or correspondence of the American Society of Civil Engineers is hereby ordered to be discontinued; and

"*Be it further Resolved*, That the appreciation and thanks of the Board of Direction of the American Society of Civil Engineers be suitably conveyed to the Standing Committee on Standard Time for their long and consistent efforts in attempting to secure the general adoption and use of the 24-hour system, an effort which, in its persistency against objections and difficulty, is none the less admirable, although failing to realize the results desired by the said Committee."

Adjourned.

October 18th, 1899.—The meeting was called to order at 8.30 P.M., Vice-President Edward P. North in the chair; Charles Warren Hunt, Secretary, and present, also, 68 members and 6 visitors.

A paper by Lewis M. Haupt, M. Am. Soc. C. E., entitled "The Reaction Breakwater as Applied to the Improvement of Ocean Bars," was presented by the author. Written discussions on the subject by Messrs. Thomas W. Symons, Elmer L. Corthell, George Y. Wisner, Henry L. Marindin and Henry C. Ripley were presented by the Secretary, and the subject was discussed orally by Mr. Thomas D. Pitts and the author.

The Secretary announced the death of Archibald Johnson, elected Member April 6th, 1898; died October 3d, 1899.

Adjourned.

*See page 211.

OF THE BOARD OF DIRECTION.

(Abstract.)

October 3d, 1899.—Vice-President North in the chair; Charles Warren Hunt, Secretary, and present, also, Messrs. Clarke, Devo, Haines, Ricketts, Schneider and Turner.

A special report on the financial condition of the Society was presented from the Finance Committee.

A progress report of a special Committee on time and place for holding the next Annual Convention was received, and the subject considered.

The matter of the propriety of providing for the payment of the expenses of members of the Nominating Committee, which was referred to the Board by the Business Meeting of the last Annual Convention, was considered, and a report adopted for presentation to the Annual Meeting.

The matter of the use of the 24-o'clock system of time notation by the Society was considered, and resolutions discontinuing its use adopted.*

The report of the Nominating Committee was received.†

Correspondence with the Director of the Twelfth Census, in regard to the collection and publication of Municipal Statistics, was presented.

Applications were considered, and other routine business transacted.

One candidate for Associate and five for Junior were elected.‡

Adjourned.

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* See page 209.

† See page 211.

‡ See page 208.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society is open every day, except Sunday, from 9 A. M. to 10 P. M.

RECENT ENGINEERING ARTICLES OF INTEREST.

In previous lists of "Recent Engineering Articles of Interest" all serial articles have been excluded. Beginning with this number of *Proceedings*, serial articles, which have been concluded within the period covered by the list, will be mentioned, and the dates when they were commenced and ended will be given.

LIST OF NOMINEES FOR THE OFFICES TO BE FILLED AT THE ANNUAL ELECTION, JANUARY 17th, 1900.

The following list of nominees for the offices to be filled at the annual election, January 17th, 1900, were presented by the Nominating Committee to the Board of Direction* at its meeting on October 3d, 1899, and announced at the meeting of the Society† on October 4th, 1899. The list has already been mailed to all Corporate Members.

For President, to serve one year.

JOHN FINDLEY WALLACE, Chicago, Ill.

For Vice-Presidents, to serve two years.

RUDOLPH HERING, New York City, representing District No. 1.
ALFRED NOBLE, Chicago, Ill., representing District No. 5.

For Treasurer, to serve one year.

JOSEPH M. KNAP, New York City, representing District No. 1.

For Directors, to serve three years.

JOHN F. O'ROURKE, New York City, representing District No. 1.
HENRY B. SEAMAN, New York City, representing District No. 1.
THOMAS H. JOHNSON, Pittsburg, Pa., representing District No. 4.
JOSEPH RAMSEY, Jr., St. Louis, Mo., representing District No. 6.
HENRY B. RICHARDSON, New Orleans, La., representing District No. 6.
GEORGE A. QUINLAN, Houston, Tex., representing District No. 7.

* See page 210.
† See page 209.

MEETINGS.

Wednesday, November 1st, 1899, at 8.30 P. M., a regular business meeting will be held. Ballots for membership will be canvassed, and a paper by Edmund B. Weston, M. Am. Soc. C. E., entitled, "Test of a Mechanical Filter," will be presented. This paper was printed in the September number of *Proceedings*.

Wednesday, November 15th, 1899, at 8.30 P. M., a regular meeting will be held, at which a paper by Frank H. Cilley, S. B., entitled, "The Exact Design of Statically Indeterminate Frameworks: An Exposition of its Possibility, but Futility," will be presented. This paper is printed in this number of *Proceedings*.

At this meeting, Mr. Wm. S. Dalrymple will address the Society on "The Triangulation of Greater New York." This address will be illustrated with lantern-slides.

Wednesday, December 6th, 1899, at 8.30 P. M., a regular business meeting will be held. Ballots for membership will be canvassed, and a paper by James A. Seddon, M. Am. Soc. C. E., entitled, "River Hydraulics," will be presented. This paper is printed in this number of *Proceedings*.

Wednesday, December 20th, 1899, at 8.30 P. M., a regular meeting will be held, at which a paper by A. H. Sabin, Assoc. M. Am. Soc. C. E., entitled, "Experiments on the Protection of Steel and Aluminum Exposed to Water," will be presented. This paper is printed in this number of *Proceedings*.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(September 13th to October 11th, 1899.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Eng. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Eng., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Can.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Indicator*, Stevens Institute, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 30c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 35c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c
(14) *The Engineering Record*, New York City, 12c
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City, 35c.
(18) *Railway and Engineering Review*, Chicago, Ill.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England.
(22) *Iron and Coal Trades Review*, London, England.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gaslight Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Electrical Engineer*, New York City, 10c.
(28) *Industries and Iron*, London, England.
(29) *Journal*, Society of Arts, London, England.
(30) *Annales des Travaux Publics de Belgique*. Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des Ecôle Spéciales de Gand*, Brussels, Belgium.
(32) *Memoirs et Compt Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mecanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Transactions*, Am. Inst. Elec. Eng., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 75c.
(45) *Mines and Minerals*, Scranton, Pa. 20c.
(46) *Scientific American*, New York City, 10c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift des Vereines Deutscher Ingenieure*, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Duesseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereines*, Vienna, Austria.
(54) *Den Tekniske Forenings Tidsskrift*, Copenhagen, Denmark.
(55) *Ingeniören*, Copenhagen, Denmark.
(56) *Teknisk Tidskrift*. Stockholm, Sweden.
(57) *Teknisk Ugeblad*. Christiana, Norway.
(58) *Proceedings*, Eng. Soc. W. Pa. 410 Penn Ave., Pittsburg, Pa. 50c.
(59) *Transactions*, Mining Institute of Scotland, London and Newcastle-upon-Tyne.
(60) *Bridges and Framed Structures*, 358 Dearborn St., Chicago, Ill., 30c.
(61) *Proceedings*, Western Railway Club. 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.

LIST OF ARTICLES.

Bridge.

The Bridge as a Religious Edifice. (14) Sept. 16.
Scherzer Rolling Lift Bridges over the Chicago River. (14) Sept. 23; (15) Sept. 22; (18) Sept. 23; (40) Sept. 22.
The Staunchness of Riveted Joints. Prof. J. T. Nicholson. (5) Part II, 1898.
Great Central Railway—Types of Girder Bridges. (11) Sept. 8.
Proposed Bridge over the Tyne. (12) Sept. 15.
Concrete-Steel Bridge Construction. Edwin Thacher, M. Am. Soc. C. E. (13) Sept. 21.
Masonry Pier Moved by Ice and Replaced. R. W. Leonard. (5) Part II, 1898.
Floor Details of a City Viaduct. (14) Sept. 30.
Railway Viaducts in Cornwall, Old and New. T. H. Gibbons. (11) Sept. 29.
The Thomson Run Bridge. (15) Oct. 6.
Mémoire sur la Construction du Pont de l'Oued Endja (Département de Constantine, Algerie). A. Daujon. (43) 1899, Second trimestre.
Note sur les Fatigues Réelles et les Fatigues Calculees dans un Pont à Grandes Mailles. M. Mesnager. (43) 1899, Second trimestre.

Electrical.

The Determination of Standards of Conductivity. Henry P. Harth. (27) Sept. 23.
On the Rational Suppression of Irrational Systems of Units. Prof. E. Hospitalier. (27) Sept. 23.
On Electric Units. Prof. E. Blondel. (27) Sept. 23.
Symbolic Representation of General Alternating Waves and of Double Frequency Vector Products. Charles Proteus Steinmetz. (42) June and July, 1899.
The Action of the Polyphase Induction Motor Graphically Explained. William Baxter, Jr. (27) Sept. 30.
Elementary Treatments of Alternating Current Problems. W. G. Rhodes. (26) Sept. 22; (20) Sept. 22.
The Equivalence of Triangles and Three-Pointed Stars in Conducting Networks. A. E. Kennelly. (27) Sept. 16.
Fly-Wheel Power of Motors. E. K. Scott. (26) Sept. 8.
Over-Compounding Dynamos. E. K. Scott. (26) Sept. 15.
Some Notes on Single-Phase Motors with Special Reference to the Heyland Motors. (26) Sept. 29.
The Electrical and Mechanical Resistance of Dynamo Brushes. (27) Sept. 16.
Electric Lighting *versus* Gas. John Martin. (28) Sept. 15.
Iron Poles, Arc Lamps and Wires in Boston. Alton D. Adams. (27) Sept. 23.
The Lighting of Lynn, (Eng.). (26) Sept. 15.
Series Arc Lighting from Constant Current Transformers. William Lispenard Robb. (13) Oct. 5.
St. Cloud, Minn., Water Power Plant. F. W. Springer. (27) Sept. 30.
Kalamazoo Valley, Mich., Transmission Plant. Edward James Hart. (27) Sept. 30.
Electrical Power Transmission from Paderno to Mailand. U. Ancona. (48) Sept. 16.
The Cost of Electricity in Some Typical Buildings in New York City. Percival Robert Moses. (42) June and July, 1899; (14) Sept. 30.
Electric Power in Engineering Works. Dr. Louis Bell. (10) Oct., 1899.
Electrical Machinery on Board Ship. Alexander Siemens. (12) Sept. 22; (47) Sept. 30.
Electrical Auxiliary Machinery in the United States Navy. Alton D. Adams. (12) Sept. 8.
Electric Light and Power Transmission at Aigle, Switzerland. (12) Sept. 8.
An Oerlikon Electric Mining Plant in Southern California. (27) Sept. 23.
The Oerlikon Engineering Works. (26) Sept. 29.
An American Electric Power Plant at the Frodingham Steel and Iron Works, England. (27) Oct. 7.
Water Power Electrical Plants in the United States. B. C. Washington, Jr. (24) Serial beginning Sept. 11, ending Sept. 18.
The Practical Application of Electric Motors to Printing Press Machinery. W. H. Tapley. (3) Oct., 1899.
Storage Batteries *versus* Electric Isolated Plants. Alton D. Adams. (10) Oct., 1899.
Distribution d'Energie par l'Electricité dans les Ateliers de Construction des Machines de la "Allgemeine Elektricitaets Gesellschaft" de Berlin. (36) Serial beginning July 10, ending Sept. 10.

Marine.

Japanese Armored Cruiser *Idzumo*. (12) Sept. 22.
H. M. S. *London*. (12) Sept. 29.
The Italian Cruiser *Garibaldi*. (11) Sept. 15.
A New Type of Battleship—The French *Henri Quatre*. (12) Sept. 8.
The Chilian Training Ship *General Baquedano*. (12) Sept. 8.
Torpedo-Boat Destroyers for Sea Service. James Howden. (47) Sept. 16.
Triple Screws for Warships. Rear Admiral George W. Melville, U. S. N. (10) Oct., 1899.
The Use of Steam in Auxiliary Machinery for Warships. Lieut. J. K. Robison. (9) Oct., 1899.
The Increasing Size of Steamships. Herbert C. Sadler. (10) Oct., 1899.
The Cunard Liner *Ivernia*. (11) Sept. 22; (12) Sept. 22.
The White Star Twin-Screw Steamer *Oceanic*. (12) Sept. 15.
The *Mabel Grace* Channel Steamer. (12) Sept. 15.

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Progress in Steam Navigation. Sir William White's Presidential Address. (11) Sept. 15; (12) Sept. 15.
Large Atlantic Cargo Steamers. G. B. Hunter. (47) Sept. 9.
Fast Cross-Channel Steamers. C. H. Parsons. (11) Sept. 22; (47) Sept. 30.
The Development of German Shipbuilding. Rudolph Haack. (9) Oct., 1899.
Tests of the Lake Steamer *Pennsylvania*, fitted with Water-Tube Boilers and Mechanical Stokers. (13) Oct. 5.
The Pendulum Propeller. H. C. Vogt. (55) Sept. 2.
Taking Steamers by the Inland Route to the Caspian Sea. (12) Sept. 22.
La Marine Marchande. J. Fleury. (32) July, 1899.

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Notes on Rod Rolling and its Development. N. K. Turnbull. (62) Serial beginning Sept. 14, ending Oct. 5.
The Niclausse Water-Tube Boiler. Mark Robinson. (12) Sept. 22; (28) Sept. 22.
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The Bauer By-Product Coke Oven. (24) Sept. 18; (52) Sept. 28.
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The Westinghouse Gas Engine. (15) Sept 15.
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Étude de la Circulation de l'Eau dans les Chaudières Multitubulaires. H. Brillié. (33) Sept. 23.
Regulateurs de Temperature. Système Dorian. (34) Sept., 1899.
Automobiles; Études sur la Direction à Deux Pivots. C. Bourlet. (33) Serial, beginning Sept. 2, ending Sept. 9.

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Deep Pumping at the Elliot Colliery. (22) Sept. 22.
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The Lake Superior Iron Ore Mines. Jeremiah Head and Archibald P. Head. (10) Oct., 1899.
Compressed Air, Electricity and Steam for Mining Purposes. W. E. Garforth. (10) Oct., 1899.
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Notes on Silver Losses in Smelting. Malvern W. Iles. (16) Serial beginning Sept. 9, ending Sept. 23.
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Pumping Difficulties in the Maypole Colliery Sinking. (22) Sept. 15.
The Lee Long-Wall Mining Machine. H. Foster Bain. (22) Sept. 22.

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Paving Brick and Brick Pavements. H. J. March. (1) August, 1899.
Distributing Gas Under High Pressure. F. H. Shelton. (13) Oct. 5.
Vitrified Clay Pipe instead of Iron Pipe for Gas Mains. Irvin Butterworth. (13) Oct. 5.

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The White Pass and Yukon Railway. (13) Oct. 5.
Steel Foundations for Steam Railroad Track. Gustav Lindenthal, M. Am. Soc. C. E. (15) Serial beginning Sept. 8, ending Oct. 6.
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Heat Transmission in Locomotive Boilers. William Forsyth. (25) Oct., 1899.
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The Various Systems of Railway Fog Signaling. Wilfrid Swanwick Boult. (21) Sept., 1899.
Le Chemin de Fer Métropolitain de Paris. J. Hervieu. (35) Serial beginning in March, ending in Sept. 1899.
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Le Transformation des Gares de Tours et de Saint-Pierre-des-Corps. MM. Couvrat and Sabouret. (38) Sept., 1889.
Note Relative à l'Emploi des Trucks-Transporteurs sur le Réseau à Voie de 0m.80 des Ardennes. M. Claise. (43) 1899, Second trimestre.
Alimentation d'eau des Locomotives en Marche. (33) Sept. 16.
Ventilation des Tunnels: Conditions d'Aeration. Volume d'Air à Introduire dans le Tunnel, et Travail Nécessaire pour son Introduction. Raymond Godfernaux. (33) Serial beginning August 19, ending Sept. 16.

Sanitary.

The Drainage Problem of New Orleans. (14) Sept. 16.
The Cleansing of Large Cities. (12) Sept. 15.
Modern Methods of Sewage Disposal. Sir William Henry Preece. (13) Sept. 14.
Improved Methods for the Purification of Sewage and Water as Shown in the Operation of the Municipal Plant at Reading, Pa. John Jerome Deery. (3) Serial beginning in Sept., ending in Oct., 1899.
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Pig-iron Fractures and their Value in Foundry Practice. James Williard Miller. (11) Sept. 22.

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Ecole de Garçons et Salle Communale de Réunion à Saint Maurice (Seine). G. Guyon. (35) Sept., 1899.
Supports Horizontaux Armaturés, en Fer et Ciment. Henry Picq. (35) Serial beginning in Aug., ending in Sept., 1899.

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The Progress of the Ordnance Survey. (12) Sept. 22.
Le Télémètre de Campagne Weldon. (36) Sept. 10.

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Report of the Commission on the Improvement of the Philadelphia Water Supply. (13) Oct. 5; (14) Oct. 7.
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Power Development at Niagara Falls other than that of the Niagara Power Company. W. C. Johnson, M. Am. Soc. C. E. (1) Aug., 1899.

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Predicting Floods in Rivers. (14) Sept. 16.
The Chittenden Drum Dam. (14) Sept. 16.
The Report of the Nicaragua Canal Commission. (13) Sept. 21.
The Soulanges Canal. Thomas Monro. (5) Part II, 1898.
Trent Canal. R. B. Rogers. (5) Part II, 1898.
Montreal, Ottawa and Georgian Bay Canal Navigation. H. K. Wicksteed. (5) Part II, 1898.
Failure of Canal Bank at Dudley Port. (12) Sept. 22.
Mechanical Traction on Canals. (12) Sept. 29
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Mémoire sur la Transformation du Canal Latéral à la Loire entre Digoin et Maimbray M. Mazoyer. (43) 1899, Second trimestre.
Notice sur la Barra de Rio Grande do Sul et sur les Moyens d'y Créer une Passe Stable. L. L. Vauthier. (43) 1899, Second trimestre.

NEW BOOKS OF THE MONTH.

Unless otherwise specified, books in this list have been donated to the Library by the Publisher.

TABLE OF SQUARES

Containing the True Square of Every Foot and Fraction Thereof from 0 to 100 feet, Advancing by One-Sixteenth of an Inch. By John L. Hall. Morocco. $5\frac{3}{4}$ x $3\frac{1}{2}$ inches, 206 pp. New York. The Engineering News Publishing Company. $2.00.

In these tables accuracy to the sixth decimal place is aimed at; the fraction being squared before it is reduced to the decimal form. The book opens with some notes on the relations of the squares, and extends the table up to 100 feet, by sixteenths of an inch. The page numbers in this book agree with the number of feet to be squared. The right-hand pages are given half numbers, with the result that all the squares pertaining to any foot are in view when the page bearing that number is referred to A separate column is devoted to each inch, broken by blanks into quarters and the roots are set in a full-face type, in order to promote clearness and ease of reference.

MASONRY CONSTRUCTION.

A Treatise on Masonry Construction. By Ira O. Baker, M. Am. Soc. C. E. Cloth, 9 x 6 ins. 556 pp. New York. John Wiley & Sons. $5.00.

This is the ninth edition of Professor Baker's work. In it the chapters on Cement, and Mortar and Concrete have been entirely rewritten and a chapter on Sand, Gravel and Broken Stone has been added. A number of minor changes have also been made. The chapter and article headings are repeated on facing pages, and there are six folding plates and an index of eight pages.

MODERN AMERICAN SCHOOL BUILDINGS.

Being a Treatise upon, and Designs for, the Construction of School Buildings. By Warren Richard Briggs. Cloth, 9 x 6 ins. 411 pp., illus. New York. John Wiley & Sons. $4.00.

This book is a rearrangement of papers prepared for State Board of Health reports and others originally written for architectual periodicals, to which new chapters, comprising nearly half the bulk of the volume, have been added. The chapter headings are: In General, Appropriations. Competitions. Specialists. Commissions, Superintendence, Ethics, Ready-Made Plans, Entrance-Halls and Staircases, Windows and Lighting, Hat-and-Cloak Rooms, Playrooms, Heating and Ventilation, Sanitary Arrangements. Hygienic Construction of the Bridgeport High School Building, Suburban Schoolhouses, Planning and Construction of Schoolhouses, City School Buildings on Restricted Sites. The book has 89 pages of illustrations, comprising elevations and floor plans of school buildings.

THE FIREPROOFING OF STEEL BUILDINGS.

By J. K. Freitag. Cloth, 9 x 6 ins. 319 pp., illus. New York. John Wiley & Sons. $2.50.

While the author recognizes that the subject of fire-proofing is still in the experimental stage, this volume is intended to present systematized information regarding the development of the fire-proofing of steel buildings and the present most approved and efficient methods of treatment. The chapter arrangements are designed to cover the development, fires and tests, materials, planning, details and equipment. There is an index of eleven pages.

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DEATH.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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PAPERS AND DISCUSSIONS.

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THE EXACT DESIGN OF STATICALLY INDETERMINATE FRAMEWORKS. AN EXPOSITION OF ITS POSSIBILITY, BUT FUTILITY.

By FRANK H. CILLEY, S. B.

TO BE PRESENTED NOVEMBER 15TH, 1899.

INTRODUCTION.

In the theoretic consideration of all material systems it is customary, in fact necessary, to replace them by more or less equivalent but simpler systems. This alone makes their analytic examination possible, not to say practicable; and the results thus obtained, although applying strictly only to the ideal constructions of theory, throw much valuable light on the workings of the actual material systems.

But so accustomed have we become to this procedure that we often forget that between the theoretic results and the actual conditions there must always exist a divergence, greater in proportion as the difference between the imaginary system and the real system is the greater. This neglect leads to two serious faults: *First*, in the consideration of existing systems, replacing them by theoretic systems, which insufficiently represent them, and applying the conclusions thus derived as though they were practically exact: *Second*, in the

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creation of new systems, constructing them so differently from the ideal systems on which they are based that the actual conditions are quite unlike those supposed. This has given rise to many strange fallacies, has held back practice and has discredited theory.

Frameworks furnish excellent illustrations of this, for the ideal framework has always been a far simpler structure than the real framework, and its theory correspondingly different from actual conditions. Whereas the ideal framework consists of a group of straight members so joined at their ends as to move freely about the points of connection, loaded only at these points, and—neglecting the effects of dead weight and wind pressure—subject, therefore, only to direct stress of tension or compression in the members, actual frameworks always have joints more or less stiff, their members are frequently joined at other points than their ends, and they are loaded at other points than their joints, thus introducing flexure of the members, as well as direct stress, as a result of the application of loads.

But it has so long been customary to overlook these divergencies between theory and practice, to gratuitously assume them of no consequence, that we now, as a matter of course, figure our simple theoretic system and then apply the results to our complex actual system, without a doubt, apparently, that this application is not only justifiable, but practically exact. This has resulted in the perpetuation of defective construction and the fallacy that increased strength and stiffness, with the same amount of material, is to be obtained by stiff joints, broad members rigidly attached where they pass one another, continuous members, etc. To this day, some eminent engineers uphold these as virtues. And, on the other hand, this has resulted in indifference as to the variations between theory and practice which actually exist, and a neglect to develop, and still more to apply, more exact parallelism, both in theory and practice. In America a partial attempt to fulfill these suppositions of the ideal framework by the use of pin connections was made long ago, yet we continue to use continuous chords, and floor systems, and many riveted connections of which the theory takes no account. In Europe a few attempts have been made to calculate closer theoretic parallels to actual structures, and these more exact calculations have shown how very considerable were the neglected elements of stress and the importance of eliminating these "secondary" stresses as they are called; but these calcula-

tions are very recent, and are extremely few in number, for they are very laborious.

The net result is, that not only do theory and practice diverge to an important extent, but that this divergence is given practically no attention. This is not as it should be, and we may look forward to considerable improvement in this respect in the near future. The author believes that a much closer approach of the actual to the ideal framework is not only practicable, but desirable; and, on the other hand, he considers more exact calculation, that is, calculation of the structures more nearly as they are, a requisite to progress. Expensive mechanical devices should not be used in the effort to approach more nearly ideal construction where no corresponding practical advantage can be shown, and laborious calculations are out of place where the limits of error of simple approximate calculations are known and can more cheaply be taken into account through liberality in construction. But what may be insisted on is that either the actual conditions shall be taken into account through more exact calculation, or that no pretence of accuracy shall be made where there has been none, and that the divergences between the theory used and the construction actually carried out shall be allowed for liberally.

The object of this paper is not to consider the more exact calculation of actual structures, but rather to examine certain questions in connection with ideal frameworks, of the greatest practical importance because at the foundation of actual designs. It deals with statically indeterminate ideal frameworks, that is, ideal frameworks as previously defined, in which not all the members are essential, or, in other words, in which some members are superfluous. What are the conditions involved and how may such frameworks be exactly designed; what are their qualities, and what their merits and demerits? Such is the matter which this paper seeks to consider. Its purpose is not to give instruction in methods, but simply to indicate the principles involved, to point out some fundamental but not generally known facts, and to discuss broadly, with the aid of a few simple illustrations, the bearing of these facts on practice. And let not such discussion of ideal construction be underrated and regarded as of no value in connection with actual structures, which all, more or less, diverge from the ideal, for it must be remembered that the ideal here considered is the basis on which all actual practice is founded.

The use of statically indeterminate frameworks, in the sense here mentioned, has been exceedingly common in Europe, where it has been, and is even now, advocated extensively by the most eminent engineers. Continuous bridges, two-hinged and hingeless arches, arches supporting continuous trusses, and their inversions of suspension type, not only are met frequently in existing structures, but are used frequently in new projects. And there are indications that here in the United States these constructions are gaining a certain hold, as shown by the erection of certain recent arches and suspension bridges, and above all by the proposed huge suspension bridge at New York. Moreover, since most frameworks are actually space-frameworks, formed by uniting several plane frameworks, and this connection is made without consideration of statical determination, most frameworks—as bridge trusses with their connecting wind bracing—are actually indeterminate, whether or not their component plane frameworks are statically determined when alone. This gives great prominence to the subject of the construction of frameworks with superfluous bars, and makes a clear understanding of them exceedingly important. Is it advisable to use such structures rather than the much simpler "just determined" construction generally advocated in America? Which is the better, and why?

In the course of the more mathematical examination which will follow, the answer to this question will present itself, but it is advisable here to review briefly certain facts.

European engineers have always been the principal advocates of indeterminate construction. They have insistently claimed for it the virtues of greater economy, strength, safety and stiffness. Practically, no demonstration of these claims has been forthcoming, except, perhaps, proof that the continuous girder possessed these advantages as compared with equal free spans; but this comparison was unfair, and the proper rejoinder to it was eventually given by the cantilever. Grudgingly conceding its equivalence, European engineers continue to claim, without demonstration, the superiority of the remaining types, particularly of the two hinged and hingeless arches over the three-hinged arch.

These claims have not been wholly uncontested. As early as 1876, Maurice Levy, in Paris, insisted strongly on the universal economic superiority of frameworks with no superfluous bars. He supported his contention by general reasoning, and demonstrated

mathematically, that in a certain exceptional case with superfluous bars the economy was no greater than without such bars. But little attention seems to have been paid to Levy's statements, possibly because they were not accompanied by any general mathematical proof. The subject, from this point of view, seems to have been dropped entirely from consideration, and the author, in his studies in Paris, Zurich and Berlin, perhaps the most notable of the European engineering schools, has found no further mention of it. In these schools statically indeterminate structures are universally taught and advocated, being regarded, apparently, as more difficult, but better, than the simpler, determined structures. Beautiful methods of calculation have been developed for the more complex structures, and the professors, possibly fascinated by the charms of these computations, gave them great prominence, and fostered among the students a predilection for difficult types, without making any study of the question, whether or not there were real advantages in their use to offset their difficulties.

Feeling convinced that, while the knowledge of methods of calculation of difficult structures was exceedingly valuable because of their frequent occurrence, and because of unavoidable occasions for their use, the encouragement, even preference, given to the use of such construction was mistaken, the author, fully informed as to European ideas and methods, undertook a general investigation of the comparative merits of frameworks with and without superfluous bars. The result was the discovery of some curiously unknown, although most fundamental, facts concerning the design of frameworks with superfluous bars and the distribution of stresses therein, together with a somewhat general demonstration of the economic superiority of statically determined over indeterminate construction. In a simple and popular article published in the *Technology Quarterly*,* for June, 1897, these results were set forth, but it was desired to present the matter, in a somewhat more mathematical and complete form, directly to the engineering public, for whom it would have a particular interest.

Consequently, the present article has been written. In it the simple illustrations and much of the matter of the preceding article have been passed over with a mere reference, the subject being treated somewhat more generally, and much space has been devoted to some illustrations of selected, extremely simple cases of exact design, with

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the purpose of demonstrating the impracticability, the futility, even the folly of attempting such designing in practice, rather than furnishing instruction in its use. It is desired simply that those who read this, by the inspection of the amount and difficulties of the work as exposed in the illustrations, may form some adequate notion of why the exact design of frameworks with superfluous bars is not to be thought of, and, therefore, why such design in practice, in the future as in the past, will be made by methods of trial, which can only give at best, after infinite labor, most imperfect and uneconomic results. But it is desired that they may examine with care the results obtained in the illustrations, to convince themselves that even with exact methods, giving the most perfect possible designs, and when neglecting temperature effects and many other serious objections, the use of superfluous bars furnishes no sensible advantage, while, including these inevitable accompaniments of indeterminate construction, it is in all respects inferior to the simple determinate construction, taught and advocated in our schools, and with which American bridge engineers are perfectly familiar.

To those whose interest and leisure permit, more careful perusal of the illustrations is recommended, and in particular the working out of similar examples. Such work would form a valuable supplement to the little that the author is here able to offer. And further, he would suggest the careful analysis of some notable existing indeterminate structures, and their comparison with designs of parallel statically determined structures. The results will inevitably be surprising in their demonstration of the sad deficiency of such indeterminate structures, judged by the standards of good proportioning insisted on in all statically determined designs. American engineers may in the future devote their attention to the determination of the best statically determined forms, to the development of statically determined space-frameworks and to the perfection of details intended to ensure the usually assumed ideal simplicity of stress, with the assurance that they are on the sole road to the best designs.

The author further ventures to hope that structures now in contemplation but not commenced, projected in statically indeterminate form, will be reconsidered, and that more economic statically determined forms, which certainly exist, will be determined and used in their stead.

The Exact Design of Statically Indeterminate Frameworks.

In an ideal framework, since it is composed only of members joined at their ends, freely movable at these joints and loaded only at the joints, we are concerned only with the equilibrium of forces acting at points—the joint centers. Therefore, the conditions at each joint are three in number, that the resultant forces along three rectangular axes shall be zero. If a framework have n joints this furnishes $3n$ equations of condition which must be satisfied. Of these $3n$ equations but $3n - 6$ are independent equations involving the inner forces alone, since they must together satisfy six conditions of equilibrium among the outer forces acting on the framework, including in these outer forces the elements of the reactions as well as the loads. In general, for the stability of the structure, these elements of the reactions must be at least six in number. Let their number be r, and the number of members of the framework or bars be m. Then, for stability, $m + r$ must at least equal $3n$. But $m + r$ may perfectly well exceed $3n$, in which case the framework becomes statically indeterminate, because the $3n$ conditions furnished by statics are insufficient to determine the $m + r$ unknown bar stresses and reaction elements.

If the framework were statically determined we should have $m + r = 3n$, the form of the framework would be just determined through the lines of its figure, it would be "just stiff"; remove any bar and it would become a mechanism; alter slightly the length of any bar or the position of any support and the figure of the framework would be altered slightly without further consequences; no stress would exist in the members, except through loads; temperature changes, whether uniform or varying, would change slightly the form of the framework without affecting it otherwise; the stress in each member due to any load would be calculable easily and without regard to its own material and its elastic qualities or those of its neighbors; each member could be designed readily and exactly under any given series of loadings to satisfy almost any given conditions, for the dimensioning of each member would be practically independent of that of all others; and, finally, all displacements of the joints of the framework due to stresses or temperature changes would be just defined through the corresponding changes of the lengths of the members, and would be easily calculable.

If to such a framework we were to add further bars, the joints

remaining as before, so that $m + r > 3n$, the framework would become overdetermined; some of the bars could be removed, and the framework would still be stiff; alter slightly the length of any of these bars or the position of a support, and the framework would be put under stress; stress without loads might, therefore, exist, and temperature changes might cause very considerable stresses; the stresses under given loads could only be calculated by long and laborious application of the equations of elasticity, in which not only the dimensions of each member but its material and elastic qualities would have to be taken into consideration; the stress in a single member could not be determined independently of other members; the design of a given member might be affected greatly by the design of many other members, and it would be very difficult, if not impossible, to satisfy accurately given conditions under a series of loadings; finally, minute changes in the lengths of some of the bars, or slight yielding of the supports, might change materially the distribution of stress, so that it would be exceedingly difficult in practice to obtain even approximately any stress distribution which in the design was determined upon, and to maintain such stress distribution in the face of any slight yielding of joints and supports due to changing loads, climate and other external, or, perhaps, also internal, causes.

Such are the structures whose exact design we are about to consider. In the foregoing we have regarded them as formed by the addition of further members to "just determined" frameworks, although they have rarely been thus conceived. This manner of regarding them is most serviceable in connection with their calculation, but the course of evolution has rather been the derivation of "just determined" forms from pre-existing indeterminate forms through the dropping of superabundant (or superfluous) members.

The recital of the more obvious characteristics of the indeterminate frameworks, just made, would seem alone a sufficient condemnation of their use in practice, and further facts, developed in the later analysis, will serve to confirm definitely this view. The statical analysis of frameworks with superfluous bars shows them to be inferior in economy and strength while not superior in stiffness to the best parallel designs without superfluous bars.

The objectionable characteristics of indeterminate structures have long been known, but hitherto have been regarded as offset by certain

supposed virtues of economy and stiffness which have been ascribed to them. Yet the correction of these mistaken beliefs through analysis, which is the object of this paper, will destroy the prestige of indeterminate structures only in part. They have, and will long continue to have, the support of many able engineers, whose experience, having been largely with such structures, prejudices them in their favor and disables them from appreciating and weighing their defects clearly. Following the common human device of seeking in reason justification for beliefs rather than through reason establishing belief, some of these engineers will make much of what analysis has not yet defined and will strive to make of the unknown, and, for the present, perhaps, unknowable, not simply a defence, but a justification of the continuance of their beliefs. We shall find them taking to ground so difficult that the theorist, at least as yet, is unable to follow and settle the matter decisively, and maintaining that, in general, the problems connected with frameworks are not static, but dynamic; that inferiority, demonstrated for static loads, no longer is true of moving loads; and that the introduction of vibration in addition to static deflections wholly alters the case and leaves indeterminate structures, after all, masters of the field.

But it must be remembered that these will only be claims, not proofs. They can only be supported by general reasoning, and can be effectively answered in the same fashion. Experience has not demonstrated that dynamic defects are necessary accompaniments of static determination, and all not proven by analysis remains disputable. Let us only not be outfaced by displays of erudition or assertions of authority, and we will speedily perceive that in appeals to unconsidered and complex phenomena (the writer has heard brought up the speed at which stress travels and the consequent effect of the time of loading on stress distribution), we have only to do with appeals to ignorance, which can hardly be regarded as adding strength to the arguments of those who make them.

The writer does not wish by the preceding to belittle the importance of dynamic phenomena. In due time and in their order they should be and doubtless will be studied and given proper consideration. But first we must study and understand the static phenomena which are at their foundation, the truths of which they will not change but only add to.

In the following paper no attempt is made to treat mathematically

other than the static side of the question. And to begin with we may point out a fact, hitherto almost wholly disregarded, and which makes indeterminate frameworks worthy of the name far more than does the fact that they are statically indeterminate.

We have already noted that in indeterminate frameworks the number of unknown quantities is greater than the number of equations of condition furnished by statics, that is $m + r > 3n$. We may conveniently regard the framework as possessing as many superfluous bars as $m + r$ exceeds $3n$. Let the bars i ($i = h$ to m inclusive) be selected as the superfluous bars; then the remaining bars e ($e = 1$ to g inclusive) form with the supports a statically determinate framework. Let S_{e_0} be the stress in any bar e of this statically determined framework if this framework alone carried the loading, let $S'_e{}^{(i)}$ be the stress in this bar e which a tension of unity in one of the superfluous bars i would cause in e if there were no external load and if of the superfluous bars the bar i alone were in action, and let S_i be the actual stress in the superfluous bar i when the given load is applied to the actual framework with all its superfluous bars acting. Then the actual stress S_e in a bar e is evidently given by the formula:

$$S_e = S_{e_0} + \sum_{i=h}^{i=m} S_i S_e{}'^{(i)}$$

Now—and here is the commonly neglected consideration—as far as the laws of statics are concerned, only the S_{e_0} and the $S_e{}'^{(i)}$ are fixed and determinate, while the S_i may be anything; they are truly indeterminate and not, as commonly stated, fixed by certain auxiliary equations of elasticity. What does fix them in a structure with given elastic constants and given bar sections is, under given conditions of temperature, the precise lengths of the bars. But, as a matter of fact, in any existing structure these lengths of bars are never known with anything like exactitude, as will be evident when one considers that, with a modulus of elasticity of 30 000 000 lbs. per square inch, a bar of steel 10 ft. long, whose length is not known within a thousandth of a foot, would not have its intensity of stress known within $\frac{30\,000\,000}{10 \times 1\,000} = 3\,000$ lbs. per square inch, or within about a third of its probable working stress. Designers and computers have passed over this difficulty, often, if not generally, unconsciously, by the simple assumption, unwarranted by any corresponding precautions in construction or knowledge of existing structures, that, at a certain

temperature and with a certain loading, a certain state of stress exists throughout the structure. Usually, this takes the simple form of the assumption that the structure, when free from all loading and uniformly at the mean temperature, is free from all stress. Then truly the equations of elasticity define the stresses under all other conditions of loading and temperature, but it is this far from warranted assumption and not the equations of elasticity which removes the indetermination.

Since the stresses in the superfluous bars are not fixed by equations of elasticity, but by their lengths, we evidently have it in our power—in designing at least—to give these bars any stresses we please under a given loading by giving them suitable lengths, and this without in any way altering the figure of the framework (because the changes of length of the superfluous bars required in all practical cases would be very small), and without altering the sections of the bars, that is, the design as it is ordinarily defined. If these superfluous bars be k in number this gives us a k^{tuple} degree of freedom in the design of such a framework, and the same degree of indetermination in the computation of such an existing framework. Unlike the case of a statically determined framework where the distribution of stresses due to a given loading is invariable, that distribution may here be varied in an infinity, a k^{tuple} infinity, of ways. In designing, we may select any one of these at will, then design the cross-sections to suit, and finally compute very exactly the corresponding lengths of the bars. It will remain to construct them exactly as computed to realize the designed stress distribution. But if the structure be an existing one given us to compute, who can say which one of this infinity of possible stress distributions is that actually existing under given conditions? The problem is actually indeterminate and in increasing degree as the number k of superfluous bars is greater.

We are now prepared to consider the subject of the title, "The Exact Design of Indeterminate Frameworks." But first, what is meant by "exact" design? Any one familiar with the design of indeterminate frameworks is aware that, unlike the design of determinate frameworks (if we neglect the estimation of dead weight), the process is essentially tentative instead of direct. That is, given the figure of the framework, certain bar sections are assumed, calculations of the stresses corresponding made (on the assumption of no stress under no load at mean temperature), the divergences from the desired con-

ditions as to intensity of stress noted, proper corrections of the sections guessed at, the computations all repeated, new corrections guessed at, then new computations made, and so on, repeating again and again all the work until the result approaches that desired sufficiently to be acceptable. But as an alteration in the section of any one bar affects the stress not in itself alone, but in many other bars also, this process of correction is not simply exceedingly laborious, but, as every designer of such structures knows, very uncertain; and after infinite labor the result obtained is usually one, the deviations of which from the set conditions of maximum allowable intensity of stress are such as would not be tolerated by any good engineer in a statically determined framework.

By "exact" design is meant: Expressing in equations the conditions as to intensity of stress to be satisfied by the solution under the given loadings, and then by direct solution of these equations, without any tentative process (other than that mathematically necessary to solve the equations), obtaining at once and exactly the desired design, *e. g.*, the cross-sections and the bar lengths. It will be appreciated that such an exact method, apart from any question of practical advantage over the old tentative method, would be likely to throw most instructive light on the properties of indeterminate designs, which the old method could only feebly give through statistics, or practically not at all; and the object in bringing forward this "exact" method is simply to throw such light; for, apart from the fact that the author reaches through this method the conclusion that the best designs of indeterminate frameworks are necessarily inferior to those of determinate frameworks which necessitate no such methods, the illustrations to follow will, among other things, show conclusively that the mathematical difficulties of the exact method render its application impracticable in general.

To begin with, we will consider the simple but very instructive and important case of design for a single given loading, subject to the condition of a given intensity of stress in each bar under that loading. This case will be somewhat briefly treated, since, in an earlier article,* it has been fully considered and illustrated.

As pointed out previously, an infinite variety of stress distributions under a given loading are all equally possible, regardless of the cross-

* *Technology Quarterly*, June, 1897. "Some Fundamental Propositions Relating to the Design of Frameworks." Those who are interested in the subject may obtain copies of this article from the Secretary, Am. Soc. C. E.

sections of the members. Having selected at will any one of these stress distributions, we next design the cross-sections of the various members to carry the stresses thus assigned them with the assigned intensities of stress, using column formulas or not as we choose. It only remains to determine the lengths of the bars necessary to ensure the supposed stress distribution. This we may do as follows:

In the given figure of the framework the exact length of every line is geometrically determinate if the data for the figure are sufficient, and this we suppose. Thus, to start with, we have what may be called the figure length L_f (where $f = 1$ to m) of every one of the m bars. Now, we know that, supposing the points of support fixed, the exact position of each joint is geometrically determined by the exact lengths from center to center of joints of the series of bars e ($e = 1$ to g), which together form a statically determined framework with the same joints (see page 573). The lengths of the bars i ($i = h$ to m), which we have termed the superfluous bars, are therefore geometrically deducible from the lengths of the bars e. Let us assume the actual unstrained lengths from center to center of joints of the bars e, at standard temperature (lengths which we may in general denote by l), to be the figure lengths, or that $l_e = L_e$. On this basis we now have to determine the actual unstrained lengths l_i from center to center of joints, at the given standard temperature, of the superfluous bars i, so that, under the given loading, each bar shall have the stress we have already determined for it.

These lengths l_i will not in general be the same as the figure lengths L_i, which we may regard as exactly defined through the L_e and therefore known. For, under the given loading the bars e are subject to the stresses S_e and, therefore, no longer have the lengths $l_e = L_e$, but have the strained lengths $L_e + \frac{S_e l_e}{E_e A_e}$, where E_e and A_e are the modulus of elasticity and the section area, respectively, of the bar e. And the distances from center to center of the joints connected by the superfluous bars i, that is the strained lengths of the superfluous bars i, are at the distances defined by and resulting from these strained lengths $L_e + \frac{S_e l_e}{E_e A_e}$. If we determine these strained lengths of the superfluous bars and then design their actual lengths l_i, so that the strained lengths shall correspond to their supposed stresses S_i, that is, so that the strained lengths shall be $l_i + \frac{S_i l_i}{E_i A_i}$ then we shall evi-

dently have so made our design as to secure our supposed distribution of stress. These strained lengths of the bars i might be directly determined by ordinary geometrical principles from the strained lengths $L_e + \frac{S_e l_e}{E_e A_e}$ of the bars e, but, since we have all the figure lengths L, and we may suppose the changes from these to be very small, it is preferable to proceed as follows, making use of the principle of virtual work. Let the bars e suffer any small elongations Δ_e from any causes whatsoever, then the consequent approach of any pair of joints not connected by a bar e will evidently be the same as the work that would be done by two equal and opposite forces of unity, each acting at those joints toward each other. through this approach, and this external work of the pair of forces unity would in turn be equal to the internal work the stresses balancing them in the bars e would do through the elongations Δ_e of the bars e. Now, these stresses in the bars e, balanciug such a pair of equal and opposite forces of unity between the joints to be connected by the superfluous bar i, we readily see are the quantities $S'_e{}^{(i)}$, already noted on page 573, so that the approach of the joints to be connected by the bar i, due to the elongations Δ_e of the bars e we at once see would be

$$\delta_i = \sum_{e=1}^{e=g} S'_e{}^{(i)} \Delta_e *$$

In our case, the Δ_e we suppose to be the elongations due to the stresses S_e, that is, we have $\Delta_e = \frac{S_e l_e}{E_e A_e}$, whence follows:

$$\delta_i = \sum_{e=1}^{e=g} \frac{S'_e{}^{(i)} S_e l_e}{E_e A_e}.$$

But $L_i - \delta_i$ is the strained length of the superfluous bar i which is also, as we have noted, $l_i + \frac{S_i l_i}{E_i A_i}$, so that we find for our desired unstrained length of the superfluous bar i the value

$$l_i = L_i - \left(\sum_{e=1}^{e=g} \frac{S'_e{}^{(i)} S_e l_e}{E_e A_e} + \frac{S_i l_i}{E_i A_i} \right)$$

$$= L_i - \sum_{f=1}^{f=m} \frac{S'_f{}^{(i)} S_f l_f}{E_f A_f}$$

(where we must note that the $S'_f{}^{(i)}$ are all zero for all values of f from h to m except i, when $S'_i{}^{(i)} = +1$).

* For a much fuller account of the principle here used and its applications, see the article "On the application of the Principle of Virtual Velocities to the Determination of the Deflections and Stresses of Frames," by Professor George F. Swain, in the *Journal* of the Franklin Institute, April, 1883. Those who are interested in the subject may obtain copies of this article from the Secretary, Am. Soc. C. E.

The foregoing neglects any temperature changes, but were there such they would be accounted for by putting $\Delta_f = \frac{S_f\, l_f}{E_f\, A_f} + \tau_f t_f\, l_f$, where τ is the coefficient of expansion and t the increase in temperature above the standard used.

This determination of the exact values of the bar lengths, which ensures the supposed stress distribution under the given loading, completes the design, but there still remains a matter of interest for consideration, the "primary" stresses or stresses remaining in the bars after the loading is removed. That such primary stresses will in general exist is evident from the fact that the superfluous bars have lengths l_i differing more or less from the figure lengths L_i, which would correspond to a condition of no stress at standard temperature with no load. Let us denote the primary stresses by (S); then, reasoning as before, the primary stresses (S_e) in the bars e determine the distance between joint centers of any one of the superfluous bars i to be

$$L_i - \sum_{e=1}^{e=g} \frac{S'_e{}^{(i)}\ (S_e)\ l_e}{E_e\, A_e}.$$

But this is the length of the bar i under its primary stress, or the length $l_i + \frac{(S_i)\ l_i}{E_i\, A_i}$; whence we obtain the equation

$$l_i + \frac{(S_i)\ l_i}{E_i\, A_i} = L_i - \sum_{e=1}^{e=g} \frac{S'_e{}^{(i)} (S_e)\ l_e}{E_e\, A_e}$$

which, adopting the previous notation, we can express in the form

$$\sum_{f=1}^{f=m} \frac{S'_f{}^{(i)}\ (S_f)\ l_f}{E_f\, A_f} = L_i - l_i.$$

Now, for each of the k superfluous bars i we can write out such an equation, each of which contains the primary stresses (S) as the only unknowns, and they appear only in the first degree. But the primary stresses (S_e) in the bars e we know are expressible in terms of the primary stresses (S_i) in the superfluous bars i through the linear equation $(S_e) = \sum_{i=h}^{i=m} (S_i)\ S_e{}'^{(i)}$ (see page 573), so that, making these substitutions in the k preceding equations, they become simply k linear equations between the k unknown primary superfluous bar stresses (S_i). These equations solved, from the now known (S_i) we determine the (S_e), and so all the primary stresses. We have only to note that the primary stresses thus found must be less than those occurring under the given loading, for the design to be acceptable.

So much for the design; now consider certain consequences. Suppose the section areas were designed by the simple rule $A = \frac{S}{\sigma}$ where σ was the working stress allowed. Then A being simply proportional to the stress S, each section area, and, therefore, the volume of material of each bar, would be simply proportional to its stress under the given load, and the total volume would be expressed simply by

$$V = \sum_{f=1}^{f=m} \frac{S_f}{\sigma_f} l_f$$

But the stresses S_e in the bars e are expressible in terms of the stresses S_{e_0}, (which would occur in them under the given loading if there were no superfluous bars), and of the arbitrarily variable superfluous bar stresses S_i, through the linear equation $S_e = S_{e_0} + \sum_{i=h}^{i=m} S_i \, S_e'^{(i)}$ (see page 573), so that ultimately the volume is expressible in the form

$$V = C + \sum_{i=h}^{i=m} c_i \, S_i$$

where c_i and C are constants. Now, were c_i and C always the same, whatever the S_i might be, V would take all values from $+\infty$ to $-\infty$ by suitably varying the S_i. But, actually, we are limited by the condition that $\frac{S_f}{\sigma_f}$ must always remain positive, so that, if by any variation of the S_i the stress in any bar becomes zero and changes to stress of the opposite kind, its working stress σ at the same instant is changed in kind, which not only involves a change in the sign of σ, but also a change in its value if different working stresses in tension and compression be used. Thus, V is expressed by a discontinuous linear function of the k independent variables S_i, the discontinuity of which occurs only when some bar stress becomes zero. Such a function has a minimum only at a point of complete discontinuity, that is, where the variation of the arbitrary stresses S_i causes k bars to pass simultaneously through zero stress, whence the conclusion:

The least material is required in a framework of given figure to resist a given loading with prescribed intensities of stress in its members, for some distribution of stress which makes zero the stress in as many bars as are superfluous.

That is:

A statically determined framework of included figure is the most economic form of a framework of given indeterminate figure for the support of a given loading.

A simple and much more objective proof of this same fact, on the same basis, due to Professor George F. Swain, will be found in the article in the *Technology Quarterly*, before mentioned. But, it may be objected that both these proofs are based on the simple rule of design $A = \frac{S}{\sigma}$, whereas posts are usually designed by more complicated formulas. Now, it can easily be shown that the change in volume of a given framework due to a slight change $d\,S_i$ in the stress in one of the superfluous bars i of the framework, independently of all others, is expressed by

$$d\,V = d\,S_i \sum_{f=1}^{f=m} \frac{S'_f{}^{(i)}\, l_f}{\phi'_f\,(A_f)}$$

where $S_f = \phi_f\,(A_f)$ expresses the relation between the section area of a bar and its stress and ϕ' is the rate of change of ϕ. Now, providing that $\phi'_f\,(A_f)$ in no case diminishes numerically as A increases, that is, providing that a post never gains less in strength for a given increase in its area as the area increases, $d\,V$ will not change its sign from negative to positive during continuous variation, and therefore changing S_i so as to make $d\,V$ negative, that is, so as to diminish the total volume of material employed, will continue to diminish that volume until a point of discontinuity is attained where some bar's section is reduced to zero. As before, we reach the conclusion that as many bars as are superfluous must have their sections reduced to zero for most economic proportions. But the condition as to post formula on which this conclusion is founded is true of all formulas of the Gordon-Rankine type, those almost universal in practice; therefore, the conclusion as to the superior economy of an included statically determined design holds, under the present standard rules of design.

We may note a further consequence, relating to stiffness. Any change in the positions of the joints, therefore the deflection of any framework, is completely defined by the strains of the bars left after excluding in any way as many bars as are superfluous (see page 576). That is, the deflection of any indeterminate framework from a state of no stress under no load (the common supposition), to its actual state of stress under a given load, is precisely the same as that of any of the determined frameworks of included figure, which we may design to carry the same loading with the same intensities of stress in the corresponding bars. But at least one of these latter frameworks would

carry this load with less material in its bars (see page 579), and therefore be stiffer construction, since, with the same amount of material in its bars, its deflections would be smaller.

Thus we see that a given loading can always be carried, with precisely the same deflections, with the same intensities of stress and with less material in the bars, by a statically determined framework the figure of which is included in that of the given indeterminate framework, than by the given framework, which may be any whatever. These same considerations, by simple modification, yield the same result as applied to the cost of the material where different materials are used, as steel cable for tension members and rolled metal for compression members.

What of the bearing of these facts? While it is true that they hold strictly only for the case of a single loading, their weight for all cases is very considerable.

"Frequently, perhaps usually in practice, some one loading or combination of loadings determines the dimensions of almost all the principal members of the framework, the members not thus determined affecting the total quantity of material employed but slightly. This is particularly the case for large structures in which dead weight plays a leading part, and where the total quantity of material employed is so large that economy in this direction is particularly desirable. In such cases one loading virtually rules the economic design, and our foregoing results hold approximately true. *The consideration of the effects of changes in loading in these cases*, although of great importance as far as the strength of the structure goes, *is of wholly secondary significance from our economic point of view.*"

With this quotation from the article in the *Technology Quarterly*, where all the foregoing will be found more fully explained as well as illustrated, we will turn to the more general and new part of the subject, the exact design of indeterminate frameworks subject to several different loadings.*

Perhaps the most readily comprehensible derivation and most simple form of the equations whose solution furnishes an exact design of any indeterminate framework of given figure subject to any given series of loadings and under any given conditions of design, is the following: As we have already noted on page 577, if, under any loading, 1, the

* The author here desires to state his indebtedness to Professor George F. Swain for the suggestion of the possibility of such exact design, and, further, to acknowledge many other valuable suggestions, as well as a critical review of the present article, by Professor Swain, under whom the author first studied structures and learned to appreciate the superiority of statically determined construction.

stresses in the members be denoted by S_{f_1} then the unstrained length of any superfluous bar i is shorter than the length of its line in the given figure of the framework by

$$\sum_{f=1}^{f=m} \frac{S_{f_1} \; S'_f{}^{(i)} \, l_f}{E_f \, A_f}$$

And if under any other loading, 2, the stresses in the same framework are S_{f_2} then the same quantity is given by

$$\sum_{f=1}^{f=m} \frac{S_{f_2} \; S'_f{}^{(i)} \, l_f}{E_f \, A_f}$$

But since the amount by which the unstrained length of any superfluous bar is shorter than the length of its line in the given figure of the framework is not, by supposition, subject to change, through change of loadings (or rather, of the stresses existing under them), we will have, for each superfluous bar i, an equation of the form

$$\sum_{f=1}^{f=m} \frac{S_{f_1} \; S'_f{}^{(i)} \, l_f}{E_f \, A_f} = \sum_{f=1}^{f=m} \frac{S_{f_2} \; S'_f{}^{(i)} \, l_f}{E_f \, A_f}$$

or, in briefer form,

$$\sum_{f=1}^{f=m} \frac{(S_{f_1} - S_{f_2}) \, S'_f{}^{(i)} \, l_f}{E_f \, A_f} = 0$$

Now, whatever our rules of design may be, the section areas A_f will be definite functions, of some sort, of the stresses S_f; and the stresses S_f are in turn linear functions of the stresses S_i in the superfluous bars through the equations

$$S_{f_1} = S_{f_{o_1}} + \sum_{i=h}^{i=m} S_{i_1} \, S'_f{}^{(i)} \qquad S_{f_2} = S_{f_{o_2}} + \sum_{i=h}^{i=m} S_{i_2} \, S'_f{}^{(i)}, \text{ etc.},$$

where the $S_{f_{o_1}}$ are the stresses that load 1 would cause in the statically determined framework composed of bars 1 to g and the $S_{f_{o_2}}$ are the corresponding stresses which load 2 would cause in that framework. Thus the equations shown are implicit equations of necessary relations between the stresses S_{i_1}, S_{i_2}, etc., in the superfluous bars i under the various loadings 1, 2, etc. For each superfluous bar i and for each pair of loadings there is one such equation, that is, if there be N different loadings, for each bar i there will be $N - 1$ such

independent equations,* and if there be k superfluous bars in all, this will give a total of $k\ (N-1)$ such independent equations between the superfluous bar stresses S_{i_1}, S_{i_2}, etc. But of these latter there are N for each bar i, that is, the stress in the bar i under each one of the N different loadings, and therefore in all there are $k\,N$ of the superfluous bar stresses S_{i_1}, S_{i_2}, etc., between which the above $k\ (N-1)$ necessary relations subsist. It follows that the values of the S_i (and therefore the design itself), possess a k^{tuple} degree of indetermination, that is, in general, k of the S_i may be arbitrarily assumed, or the stress in each superfluous bar under one loading may be arbitrarily taken, or, more generally, any k further consistent relations between the stresses S_i may arbitrarily be set, and then a solution rigorously satisfying the given conditions of design as expressed through the primary equations, directly obtained. For, the solution of the equations under these circumstances furnishes us the stresses S_i in the superfluous bars under all the given loadings, thence at once all the stresses under the given loadings, thence through the prescribed conditions of design all the section areas, and finally thence, as previously explained, the precise unstrained lengths of the superfluous bars. But here a criticism is necessary. The complete mathematical solution of such a set of equations will, in general, consist of a considerable number of separate solutions, not all of which may be real, and of the real solutions, some or all of which may be inconsistent with conditions of design not expressible in the equations. These will generally be conditions of inequality such as that in the design thus obtained the most unfavorable demands shall be those supposed most unfavorable in drawing up the equations, and therefore the basis of those equations. For example, the rule of design being that each bar under its most unfavorable loading shall have stress of a prescribed intensity, in drawing up the equations, the most unfavorable loading for each bar must, in advance, be judged, the equations drawn up accordingly, and then the solutions thus obtained, which are real (and which will necessarily satisfy the condition that under the selected loading for each bar it has the prescribed intensity of stress), must further be tested as to their satisfying the suppositions as to most unfavorable loadings under which the equa-

* If two extremes of temperature were also to be considered, the effect would be to double the number of loadings. Temperature may be taken into consideration by putting $\frac{S_f}{E_f A_f} + \tau_f t_f$ for $\frac{S_f}{E_f A_f}$ where τ_f is the coefficient of expansion.

tions were drawn up. Frequently, none of the real mathematical solutions, where these exist, will satisfy these conditions, and thus limitations arise as to the arbitrary setting of stresses in the superfluous bars, or more generally, the arbitrary setting of k additional relations between these stresses. Nevertheless, a k^{tuple} infinity of designs of an indeterminate framework of given figure, under given loadings and subject to given conditions of design, is in general possible. It remains for us first to note some of the consequences, and second, to illustrate them by simple examples.

One will be most impressed, perhaps, by the fact of the possible multiplicity of exact and admissible designs of frameworks of given figure subject to given loadings. Clearly, any one of these designs will not necessarily be a good design, and, therefore, the fact that a design for an indeterminate framework of given figure subject to given loadings, rigorously fulfills the prescribed conditions of design is no indication whatsoever that the design in question is to be approved. Some such designs will be far more economic than others, that is, will require far less material; some such designs will be far stiffer than others, or will have other points of superiority. Our end is not simply to obtain any such design, but the best of such designs.

But what is the best such design? Suppose we say that design which is most economic of material is the best. The material in the bars is expressed by

$$\sum_{f=1}^{f=m} A_f l_f$$

which is a function, through the prescribed conditions of design, of the bar stresses S_f, that is, finally, of the superfluous bar stresses S_i. We may determine mathematically the necessary relations for this to be a minimum, make these the additional arbitrary relations and thus obtain the most economic solutions. The most economic of these economic designs will be that sought. Theoretically the way is clear. And what will be the nature of this most economic design? No longer, as in the case of design under a single loading, will it necessarily be some statically determined framework of included figure. The fact that the volume of material no longer is expressible on any basis of design as a linear function of independent variables would alone

suggest this, and illustrations may easily be found which conclusively prove it. But, as will later be shown, this possible superiority of an indeterminate design, which will only occur when a structure is subject to very diverse principal loadings, and not at all, or in but small degree, to large structures, the dead weight of which is prominent, is never likely to be considerable, and only exists at all through the neglect of temperature stresses to which the indeterminate structure is subject where the determinate structure is not, and the further neglect of all consequences of dimensioning and work in erection of other than the most extreme accuracy, the neglect of any lack of absolute resistance of all supports, and the neglect of any yielding at the joints and set or other yielding under operation; all of which must in practice exist, and considerably affect the indeterminate construction where they inappreciably affect the determinate. Finally, simply as an interesting although perhaps not practically important fact, it may be observed that by a combination of statically determined designs, of figures included within that of a given indeterminate framework, or what the author calls a multiple design, a series of different loadings can always be carried more economically with determinate stresses than by even the most economic of single indeterminate designs of the given figure, so that the conclusion is absolutely warranted that:

"Statical indetermination in a structure is always to be regarded as self-interference with efficiency."

Turning to the question of stiffness, but little can be added to what has already been said. Indeterminate designs subject to several loadings will in general have deflections of a different character from those of any one of the determinate designs of included figure. It is, therefore, impossible to set any basis of comparison that would not at once be subject to much criticism. The writer will merely observe that the comparisons between such structures, with which he is familiar, have usually been made on most unfair bases, and some too generally accepted conclusions, notably on the comparative stiffness of three-hinged and two-hinged or hingeless arches, are not justifiable. The writer's own very limited but fairly equable comparisons show clearly that no very marked superiority can be attributed to either type in this respect, and that points of superiority in one are offset by other

points of superiority in the other, so that nothing but a debatable conclusion in any case is possible. Some later illustrations will make this clear. And in fairness it must be conceded that the fact that in certain cases, in the attempt to secure static determination through the use of certain devices, play has been introduced most objectionable in connection with loads moving at high speed, only shows the defective character of the devices used, and in no wise attains the principle the application of which does not necessarily involve the use of these devices.

Lastly, what of the superior safety of frameworks with superfluous bars? Since there are superfluous bars there are bars the rupture of which does not necessarily involve the failure of the framework, and this is certainly an element of safety; but, generally, not every bar may be regarded as a superfluous bar, so that in an indeterminate framework the rupture of some bars would not less necessarily involve the failure of the structure than in the case of a determinate framework. Finally, even of the bars the omission of which would leave the remaining portion statically determined, and, therefore, which, from this point of view, are not essential, some, perhaps many, will be of such actual importance in the structure that their removal would result in serious overstrain of some of the remaining bars, and thereby failure; while of those still remaining the sudden rupture of some and the accompanying shock would be fatal to the structure. The superior safety due to the use of superfluous bars is evidently very limited; and, when we note that the statically determined structures require less material to satisfy the same conditions of design, it becomes questionable whether for equal amounts of material the determinate design, which would then have lower unit stress than the indeterminate design, would not, therefore, be at least as safe.

Turning from these general considerations let us examine a little more closely the equations through the solution of which exact designs are to be obtained. The implicit and symbolic form in which we have stated them gives them a most deceptive appearance of simplicity which is very far from corresponding with facts. In explicit form we should find each equation to consist of a series of terms in the form of fractions, each fraction having for numerator a linear function of the variables S_i, but for denominator a linear or

higher degree function of the variables, according to the complexity of the rules of design used, and, very likely, it would not be a whole integral function. The simplest possible rule of design, making the section areas simply proportional to the greatest stress borne—which is the common rule for tension members—makes the denominator linear in terms of the variables S_i. But the introduction of post formulas of the Gordon-Rankine type, now generally used, at once would introduce fractional functions of the variable into the denominator. Now, these equations in fractions have a most delusive appearance of simplicity. Only when we have accomplished their reduction to whole integral equations, a very considerable although theoretically simple undertaking in itself, are we able to form some conception of what is involved in obtaining a solution. Then we discover that we have a series of simultaneous equations of higher degree than the first between numerous unknowns. The indetermination having been removed by the introduction of suitable arbitrary relations, by perfectly well-known methods of elimination, these equations are eventually reducible to a single equation with one unknown, the solution of which then by numerical methods (for, of course, we are dealing with equations with known numerical coefficients), offers no theoretic difficulty; but practically in any actual case the work involved would be appalling. And then the solutions obtained may prove worthless, as they may not be consistent with the suppositions on which the equations were drawn up. The number of such possible suppositions will generally be very considerable, and it may well happen that many trials will be necessary before admissible solutions will be found, thus multiplying the labor many times; and, if we wished to be complete in our mathematical solution, all the possible suppositions should be examined, making an undertaking from which even the bravest calculator might well recoil.

In order to give a conception, even though faint, of the appearance of one of the primary equations when written explicitly, but simply and in fractional form, such an equation for the bar i, when there are only the two different loadings 1 and 2, and when the basis of design is simply $A = \frac{S}{\sigma}$, is given, as follows:

$$\sum_{b=1}^{b=c} \frac{\sigma_b (S_{b_{0_1}} - S_{b_{0_2}})\, S'_b{}^{(i)} l_b}{E_b (S_{b_{0_1}} + \sum_{i=h}^{i=m} S_{i_1} S'_b{}^{(i)})} + \sum_{e=d}^{e=g} \frac{\sigma_e (S_{e_{0_1}} - S_{e_{0_2}})\, S'_e{}^{(i)} l_e}{E_e (S_{e_{0_2}} + \sum_{i=h}^{i=m} S_{i_2} S'_e{}^{(i)})}$$

$$+ (S_{h_1} - S_{h_2}) \left[\sum_{b=1}^{b=c} \frac{\sigma_b S'_b{}^{(h)} S'_b{}^{(i)} l_b}{E_b (S_{b_{0_1}} + \sum_{i=h}^{i=m} S_{i_1} S'_b{}^{(i)})} + \sum_{e=d}^{e=g} \frac{\sigma_e S'_e{}^{(h)} S'_e{}^{(i)} l_e}{E_e (S_{e_{0_2}} + \sum_{i=h}^{i=m} S_{i_2} S'_e{}^{(i)})} \right]$$

$$+ (S_{(h+1)_1} - S_{(h+1)_2}) \left[\sum_{b=1}^{b=c} \frac{\sigma_b S'_b{}^{(h+1)} S'_b{}^{(i)} l_b}{E_b (S_{b_{0_1}} + \sum_{i=h}^{i=m} S_{i_1} S'_b{}^{(i)})} + \sum_{e=d}^{e=g} \frac{\sigma_e S'_e{}^{(h+1)} S'_e{}^{(i)} l_e}{E_e (S_{e_{0_2}} + \sum_{i=h}^{i=m} S_{i_2} S'_e{}^{(i)})} \right]$$

$$+ \cdots\cdots\cdots\

cases for investigation would have been further and greatly increased. So it will be seen that a complete, exact, mathematical solution, even on the simplest possible conditions of design, involves great labor, very rapidly increasing as the number of bars in the framework increases. The justness of the statement, made earlier, that the exact method was not to be thought of for use in practical designing, will now be perceived, and it is thought that the illustrations following will amply confirm this. But since exact methods are not applicable in practice, and the tentative methods of the past will continue to be used, and at best can never give better than most imperfect results, the impossibility of benefiting through the use of indeterminate designs, of even their theoretically possible advantages, will be clear; a final and fatal fact for the adherents to such construction. If ever an ideal has been completely vindicated by increased knowledge it is certainly that of the statically determined construction of frameworks.

Illustrations of Design of Indeterminate Frameworks under Multiple Loadings.

Let it be required to design a framework of the form and dimensions shown in Fig. A, so that it shall support either

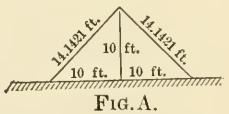


Fig. A.

the pull P in a horizontal direction, thus Fig. B. or the weight W, thus Fig. C. so that each bar, under its most unfavorable load, shall have stress of the mean intensity of 5 tons per square inch. Denoting the diagonals by the numbers 1 and 2, respectively, and the vertical Fig. D. by the number 3 (see Fig. D), and calling the loading with the horizontal pull the loading P, and the loading with the vertical weight the loading W, let us suppose that in the bars 1 and 2 the maximum stress occurs under the loading P and in the bar 3 under the loading W, so that

$$A_1 = \frac{S_{1_P}}{+5} \qquad A_2 = \frac{S_{2_P}}{-5} \qquad A_3 = \frac{S_{3_W}}{-5}$$

the bar 1 being designed under tensile stress and the bars 2 and 3 under compression.

Treat bar 3 as the superfluous bar. Then unity tension in it would cause compression of 0.707 in each of the bars 1 and 2 or $S'_1 = S'_2 = -0.707$ (see Fig. E). The apex of the triangle sinks more under the load W than under the pull P by

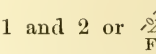


Fig. E.

$$\frac{S'_1\,(S_{1_W} - S_{1_P})\,l_1}{EA_1} + \frac{S'_2\,(S_{2_W} - S_{2_P})\,l_2}{EA_2} = (S_{3_P} - S_{3_W})\,\frac{l_3}{EA_3}$$

and, noting the values of S', l and A, this gives the simple relation

$$\frac{S_{1_P} - S_{1_W}}{S_{1_P}} + \frac{S_{2_P} - S_{2_W}}{-S_{2_P}} = \frac{S_{3_P} - S_{3_W}}{-S_{3_W}}$$

Now, $S_{1_P} = S_{1_{0_P}} + S'_1\,S_{3_P} = +0.707P - 0.707S_{3_P}$

$$S_{1_W} = S_{1_{0_W}} + S'_1\,S_{3_W} = -0.707\,W - 0.707S_{3_W}$$

$$S_{2_P} = S_{2_{0_P}} + S'_2\,S_{3_P} = -0.707P - 0.707S_{3_P}$$

$$S_{2_W} = S_{2_{0_W}} + S'_2\,S_{3_W} = -0.707\,W - 0.707S_{3_W}$$

so that the relation may be written

$$\frac{P + W - S_{3_P} + S_{3_W}}{P - S_{3_P}} - \frac{P - W + S_{3_P} - S_{3_W}}{P + S_{3_P}} = \frac{S_{3_W} - S_{3_P}}{S_{3_W}}$$

or, clearing of fractions,

$$2\,P\,S^2_{3_W} + (2\,P\,W - P^2 + S^2_{3_P})\,S_{3_W} + P^2\,S_{3_P} - S^3_{3_P} = 0$$

which is a quadratic equation in S_{3_W} and a cubic in S_{3_P}. We observe the following relations:

If $S_{3_W} = 0$, then $S_{3_P} = 0$, the bar 3 having zero area.

If $S_{3_P} = \pm P$, then $S_{3_W} = -W$, either bar 1 or bar 2 having zero area.

If $S_{3_P} = 0$, then $S_{3_W} = -W + \frac{P}{2}$, bars 1 and 2 being of the same area and bearing together $\frac{P}{2}$, whereby their junction sinks the full allowable strain of 3.

We note that S_{3_P} is the primary stress in bar 3, since it does not vary with P, and the primary stresses in 1 and 2 are $-0.707\,S_{3_P}$.

It will be seen that the bar 3 can only be used to advantage when W is greater than P, which we will suppose to be the case.

Since 1 and 2 are most efficient in resisting P we have supposed them designed under P, while 3, which is only efficient in resisting W,

we have supposed designed under W. Thus we must satisfy the inequalities

$$S^2_{1_P} \geqq S^2_{1_W} \qquad S^2_{1_P} \geqq \frac{S^2_{3_P}}{2}$$

$$S^2_{3_W} \geqq S^2_{3_P} \text{ and also}$$

$$S^2_{2_P} \geqq S^2_{2_W} \qquad S^2_{2_P} \geqq \frac{S^2_{3_P}}{2}$$

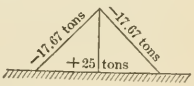
The latter pair of inequalities are necessitated by the primary stresses.

A consequence of this last pair of inequalities is that $S^2_{3_P} \leqq \left(\frac{P}{2}\right)^2$ so that $\frac{(S_{1_P} - S_{2_P})\, l_1}{5}$ is the volume of material in the bars 1 and 2 under these circumstances, and is constant. For economy then, we must so choose S_{3_P} within the above limits that, while satisfying the remaining inequalities, $S^2_{3_W}$ shall be as small as possible.

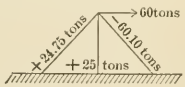
Let us take $W = 100$ tons, $P = 60$ tons, and assume $S_{3_P} = 25$ tons. We have $S^2_{3_W} + \frac{9025}{120} S_{3_W} + (750 - \frac{15625}{120}) = 0$, whence $S_{3_W} = -37.6 \pm 28.2 = -9.4$ tons, or -65.8 tons, of which the former is excluded by the condition $S^2_{3_W} \geqq S^2_{3_P}$.

Taking the solution of $S_{3_W} = -65.8$ tons, we obtain the following:

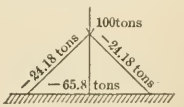
With no load, $S_1 = S_2 = -17.67$ tons, $S_3 = +25$ tons.



With load $P = 60$ tons, $S_{1_P} = +24.75$ tons, $S_{2_P} = -60.10$ tons, $S_{3_P} = +25$ tons.



With load $W = 100$ tons, $S_{1_W} = -24.18$ tons, $S_{2_W} = -24.18$ tons, $S_{3_W} = -65.8$ tons.



The section areas are: (bar 1) $\frac{24.75}{5} = 4.95$ sq. ins., (bar 2) $\frac{-60.10}{-5} = 12.02$ sq. ins., and (bar 3) $\frac{-65.8}{-5} = 13.16$ sq. ins.

The volumes are: (bar 1) 840.0 cu. ins., (bar 2) 2 039.8 cu. ins., (bar 3) 1 579.2 cu. ins., a total of 4 459 cu. ins.

Consider the following comparisons:

The statically determined form has areas (bar 1) 16.97 sq. ins., (bar 3) 20 sq. ins.; and volumes (bar 1) 2 879.8 cu. ins., (bar 3) 2 400 cu. ins., a total of 5 279.8 cu. ins.

The statically determined form has areas (bars 1 and 2) each 14.14 sq. ins.; and volumes (bars 1 and 2) each 2 400 cu. ins., a total of 4 800 cu. ins.

But the combination or "multiple" form for P and for $W - P$ has areas (bars 1 and 2) each 8.48 sq. ins., and (bar 3) 8 sq. ins.; and volumes (bars 1 and 2) each 1 440 cu. ins., (bar 3) 960 cu. ins., a total of but 3 840 cu. ins.

This illustrates clearly how the indeterminate form may be more economic than any single included determinate form, yet not as economic as a combination of determinate forms.

The second value of S (— 9.4 tons), excluded by the condition $S^2_{3_P} \geqq S^2_{3_W}$, appears in the solution correctly as one of the answers to the unrestricted problem. This partial exclusion of solutions, as previously noted, would usually occur in problems of this description, and often not even one of the solutions would be admissible under the limitations prescribed.

Thus, in this case, S_{1_P} being tension and S_{1_W} being compression, the condition $S^2_{1_P} \geqq S^2_{1_W}$ may be written $S_{1_P} \geqq -S_{1_W}$; whence $P - S_{3_P} \gtreqless W + S_{3_W}$. The limiting value of this relation is $P - W - S_{3_P} = S_{3_W}$, or, in this case, $40 + S_{3_P} = -S_{3_W}$. If we substitute this in the equation

$$\frac{P + W - S_{3_P} + S_{3_W}}{P - S_{3_P}} - \frac{P - W + S_{3_P} - S_{3_W}}{P + S_{3_P}} + \frac{S_{3_P} - S_{3_W}}{S_{3_W}} = 0$$

this becomes $2 - \frac{2\,S_{3_P}}{60 + S_{3_P}} - \frac{40 + 2S_{3_P}}{40 + S_{3_P}} = 0$; or $S^2_{3_P} + 20S_{3_P} - 1\,200 = 0$, a quadratic, of which $S_{3_P} = +\,26.056$ is one root, and the corresponding value of S_{3_W} is $-\,(40 + S_{3_P}) = -\,66.05$. If we were to assume S_{3_P} greater than + 26.056, there would be no admissible solution.

At first glance it would seem that the greater S_{3_P}, the smaller (numerically) would be S_{3_W}; but if we differentiate the equation of

relation between S_{3P} and S_{3W} and put $\frac{dS_{3W}}{dS_{3P}} = 0$, we obtain the condition

$$S_{3W} = \frac{3}{2}\left(\frac{S^2_{3P} - \frac{1}{3}P^2}{S_{3P}}\right) \text{ or, in this case, } S_{3W} = \frac{3}{2}\left(\frac{S^2_{3P} - 1\,200}{S_{3P}}\right),$$

and substituting this in the equation of relation between S_{3P} and S_{3W}, we obtain the quintic

$$S^5_{3P} + 540S^4_{3P} + 28\,800S^3_{3P} - 1\,296\,000S^2_{3P} - 30\,240\,000S_{3P} + 777\,600\,000 = 0.$$

One root is $S_{3P} = +19.168$, and the corresponding (minimum) value of S_{3W} is $S_{3W} = -65.14$, which gives only slightly more economy than $S_{3P} = +25$.

Thus far we have examined only the case of bars 1 and 2 with maximum loads under P, and bar 3 with maximum load under W, and, moreover, the suppositions $S_{1P} > 0$, $S_{2P} < 0$, $S_{3W} < 0$. These latter were not, perhaps, necessary, but apparently are essential to economic solutions. On the other hand, although the argument that the bars 1 and 2 should have their maximum loads under P for a most economic design looks reasonable, it may be misleading, so let us examine some further cases.

As a second case suppose bar 1 to have its maximum stress under W (instead of under P), bar 2 having its maximum stress under P, and bar 3 its maximum stress under W as before. Here we will suppose $S_{1W} < 0$, $S_{2P} < 0$ and $S_{3W} < 0$, so that the bars are all designed under compression. The equation is

$$\frac{S_{1P} - S_{1W}}{-S_{1W}} + \frac{S_{2P} - S_{2W}}{-S_{2P}} = \frac{S_{3P} - S_{3W}}{-S_{3W}}$$

or, replacing S_{1P}, S_{1W}, S_{2P} and S_{2W} by their values in P, W, S_{3P} and S_{3W},

$$\frac{P + W - S_{3P} + S_{3W}}{W + S_{3W}} - \frac{P - W + S_{3P} - S_{3W}}{P + S_{3P}} + \frac{S_{3P} - S_{3W}}{S_{3W}} = 0,$$

which, cleared of fractions, gives

$$W S^2_{3P} + (P\ W + P\ S_{3W} - W\ S_{3W} - S^2_{3W})\ S_{3P} +$$
$$(P^2 - P\ W + W^2)\ S_{3W} - (P - 2\ W)\ S^2_{3W} + S^3_{3W} = 0,$$

which is a quadratic in S_{3P} and a cubic in S_{3W}. Now, we have in this case

$$A_1 = \frac{S_{1W}}{-5} = \frac{0.707\ (W + S_{3W})}{5} \text{ sq. ins.} \qquad l_1 = 12 \times 14.14 \text{ ins.}$$

$$A_2 = \frac{S_{2P}}{-5} = \frac{0.707\ (P + S_{3P})}{5} \text{ sq. ins.} \qquad l_2 = 12 \times 14.14 \text{ ins.}$$

$$A_3 = \frac{S_{3W}}{-5} = \frac{-S_{3W}}{5} \text{ sq. ins.} \qquad l_3 = 12 \times 10 \text{ ins.}$$

$$\text{Total volume} = A_1 l_1 + A_2 l_2 + A_3 l_3 = 24\,(P + W + S_{3P}).$$

It is clear that for economy we must aim to make S_{3P} as small as possible while satisfying the inequalities $S^2_{1W} > S^2_{1P}$, $S^2_{2P} > S^2_{2W}$, $S^2_{3W} > S^2_{3P}$. The first two of these conditions combined require $S^2_{2P} > S^2_{1P}$, since $S_{1W} = S_{2W}$ necessarily; and thence follows that $S_{3P} > 0$. Since $S_{2P} \leqq 0$, $-S_{2P} \geqq -S_{2W}$, or $P - W + S_{3P} - S_{3W} \geqq 0$. If $P + S_{3P} = W + S_{3W}$, then $\dfrac{P + W - S_{3P} + S_{3W}}{W + S_{3W}} + \dfrac{S_{3P} - S_{3W}}{S_{3W}} = 0$;

whence $S_{3P} = P\ \dfrac{W - P}{W + P} = +15$ tons, $S_{3W} = -25$ tons.

Now, $S_{3P} - S_{3W}$ diminishes with S_{3P} at these values, and they alone satisfy $P + S_{3P} - W - S_{3W} = 0$. Thus, if $P + S_{3P} - W - S_3 \geqq 0$, then $S_{3P} \geqq P\ \dfrac{W - P}{W + P}$. At the limit we may take $S_{3P} = P\ \dfrac{W - P}{W + P}$ $= 15$ tons, and this is the least allowable value of S_{3P} in this case. The corresponding values are:

$S_{3W} = P - W + S_{3P} = -25$ tons,

$S_{2P} = S_{2W} = S_{1W} = -0.707\ (P + S_{3P}) = -53.02$ tons,

$S_{1P} = 0.707\ (P - S_{3P}) = +31.81$ tons,

$A_1 = A_2 = 10.60$ sq. ins.,

$A_3 = 5$ sq. ins.,

Vol. bar 1 = vol. bar 2 = 1 800 cu. ins.,

Vol. bar 3 = 600 cu. ins.

Total volume of bars = 4 200 cu. ins., which is a considerably more economic design than that of our first case. We note that, since $A_1 = A_2$ $S_{3P} = +15$ tons is also the primary stress in bar 3.

As a third case, suppose bar 1 has its maximum stress under P, but bar 2 has its maximum stress under W, and bar 3 has its maximum stress under W as before. In this case $S^2_{1P} > S^2_{1W}$, $S^2_{2W} > S^2_{2P}$, $S^2_{3W} > S^2_{3P}$.

Since $S_{1_W} = S_{2_W}$, $S^2_{1_P} > S^2_{2_P}$; whence $S_{3_P} < 0$, $S_{1_P} > 0$ and $S_{3_W} < 0$. This latter follows from $S_{1_P} > -S_{1_W}$, or $P - S_{3_P} > W + S_{3_W}$ and $S^2_{3_W} > S^2_{3_P}$.

Now, if we suppose $S_{2_W} = S_{1_W} < 0$, then the equation would be

$$\frac{S_{1_P} - S_{1_W}}{S_{1_P}} + \frac{S_{2_P} - S_{2_W}}{-S_{2_W}} = \frac{S_{3_P} - S_{3_W}}{-S_{3_W}}$$

which we may write $1 - \frac{S_{1_W}}{S_{1_P}} - \frac{S_{2_P}}{S_{2_W}} + \frac{S_{3_P}}{S_{3_W}} = 0$. Since the terms following the first are numerically less than unity by supposition, this equation asserts that $1 + (>0) \pm (<1) + (>0) = 0$, which is impossible.

If we suppose $S_{2_W} = S_{1_W} > 0$, then the equation would be

$$\frac{S_{1_P} - S_{1_W}}{S_{1_P}} + \frac{S_{2_P} - S_{2_W}}{S_{2_W}} = \frac{S_{3_P} - S_{3_W}}{-S_{3_W}}$$

which we may write $-1 - \frac{S_{1_W}}{S_{1_P}} + \frac{S_{2_P}}{S_{2_W}} + \frac{S_{3_P}}{S_{3_W}} = 0$

or $-1 - (>0) \pm \left(\begin{matrix}<1\\>0\end{matrix}\right) + (<1) = 0$, which is possible only if $S_{2_P} > 0$, that is, if $P + S_3 < 0$, or if $S_{3_P} < -P$. Since $S_{2_W} > 0$, $-(W + S_{3_W}) > 0$, or $S_{3_W} < -W$. Now, $A_1 = \frac{0.707\,(P - S_{3_P})}{5}$, $A_2 = \frac{-0.707\,(W + S_{3_W})}{5}$, $A_3 = \frac{-S_{3_W}}{5}$, $A_1 l_1 + A_2 l_2 + A_3 l_3 = 24\,(P - W - S_{3_P} - 2\,S_{3_W})$ cu. ins. This will be a minimum for $S_{3_P} + 2\,S_{3_W}$ a maximum. But the maximum values $S_{3_P} = -P$ and $S_{3_W} = -W$ occur simultaneously, making $S_{3_P} + 2\,S_{3_W} = -(P + 2\,W)$ a maximum, therefore, we have

$S_{3_P} = -60$ tons,	$A_1 = 16.96$ sq. ins.,	Vol. bar $1 = 2\,880$ cu. ins.,
$S_{3_W} = -100$ tons,	$A_2 = 0$,	Vol bar $2 = 0$,
$S_{1_P} = +84.84$ tons,	$A_3 = 20$ sq. ins.,	Vol. bar $3 = 2\,400$ cu. ins.
$S_{2_P} = S_{1_W} = S_{2_W} = 0$,		

Total volume, $= 24\,(60 - 100 + 60 + 200) = 5\,280$ cu. ins.

This "economic" solution is a statically determined form.

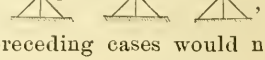
As a fourth case let all the bars have their maximum stresses under W, then clearly $S_{1_W} < 0$, $S_{2_W} < 0$, and the equation is

$$\frac{S_{1_P} - S_{1_W}}{-S_{1_W}} + \frac{S_{2_P} - S_{2_W}}{-S_{2_W}} = \frac{S_{3_P} - S_{3_W}}{-S_{3_W}}$$

Here $A_1 = A_2$, and consequently S_{3_P} is the primary stress in 3. $A_1 = \frac{S_{1_W}}{-5} = \frac{0.707\,(W + S_{3_W})}{5} = A_2$, $A_3 = \frac{-S_{3_W}}{5}$, $A_1 l_1 + A_2 l_2 + A_3 l_3 =$ 24 $(2\,W + S_{3_W})$ cu. ins. This will be a minimum for S_{3_W} a minimum. Now, from $S^2_{2_W} > S^2_{2_P}$ follows $W + S_{3_W} > P + S_{3_P}$, or $S_{3_W} > P - W + S_{3_P}$, or $W + S_{3_W} - S_{3_P} > P$. But $\frac{S_{1_P} - S_{1_W}}{-S_{1_W}} + \frac{S_{2_P} - S_{2_W}}{-S_{2_W}} + \frac{S_{3_P} - S_{3_W}}{+S_{3_W}} = 0$ gives $(W + S_{3_W} - S_{3_P})\,S_{3_W} + W\,S_{3_P} = 0$; whence $P\,S_{3_W} + W\,S_{3_P} > 0$, since $S_{3_W} < 0$. This gives $S_{3_P} > -\frac{P}{W}\,S_{3_W}$; whence $S_{3_W} > P - W - \frac{P}{W}\,S_{3_W}$, or $S_{3_W} > W\,\frac{P - W}{P + W}$, or $S_{3_W} > -25$. At limit $S_{3_W} = -25$ tons (minimum), $S_{3_P} = +15$ tons, $S_{1_P} = +31.81$ tons, $S_{2_P} = S_{2_W} = S_{1_W} = -53.02$ tons; $A_1 = A_2 = 10.60$ sq. ins., $A_3 = 5$ sq. ins. Total volume $= 24\,(200 - 25) = 4\,200$ cu. ins., which is identical with the most economic design of the second case.

We might consider further cases in which bar 3 was designed under the load P, but it seems sufficiently evident that these could not be economic designs and need no study.

The final conclusion is that in the most economic design all bars have their maximum stresses under the vertical load, and one of the bars has the same stress under the horizontal pull, a result somewhat different from that anticipated at the outset.

As a last study, suppose it were required that the frame should support any one of the three loadings  , then solutions of the first three of these preceding cases would not in general apply, since they are non-symmetrical. But solutions of the last of the cases preceding evidently would apply, since they are symmetrical, and the most economic solution will be the same here as there. That is to say, the design with $A_1 = A_2 = 10.60$ sq. ins., $A_3 = 5$ sq. ins., and having a volume of 4 200 cu. ins., and a primary stress in the vertical of $+15$ tons, is the most economic design for the three loadings as shown. This is as economic as the best design for the smaller number of loadings, but usually the economic design for the smaller number of loadings will be the more economical, and, evidently, it will never be the less economical.

This example, perhaps the simplest that could be selected in the design of indeterminate frameworks subject to multiple loadings, will furnish a conception of the difficulty, or rather the practical impossibility, of any general application of such analysis. Here, for single solutions independent of limiting conditions, the solution of quadratic equations was necessary, in one of the cases the solution of a quintic was involved in the determination of economic proportions, and last, but perhaps not least, the examination of numerous cases of defining loadings together with puzzling questions in connection with limiting conditions, had to be gone through with before the most economic design was found and established as such. In any other case the work of this case (which is hardly more than outlined here), and the difficulties would be multiplied many fold. The equations would be of much higher degree and simultaneous, the different cases of defining loadings to be examined would be far more numerous, and the consequences of the limiting conditions would be infinitely more complex and difficult to trace out. In problems of a practical description the quantity and complexity of the work would be such that it is safe to regard it as prohibitive.

Illustration of the Design of an Indeterminate Arch Under Several Loadings.

As a second example consider the two-hinged arch shown in Fig. 1.

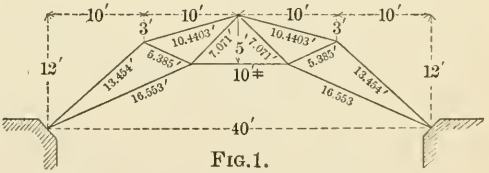


Fig. 1.

The dimensions of the arch are as follows: Span, 40 ft., divided into four parts of 10 ft. each by the upper chord, the rise being 12 ft., and the quarter points being on the parabola through the vertex and the supports. The middle tie is 10 ft. long and 5 ft. from the vertex and all other dimensions follow as shown.

The arch being symmetrical, it is most desirable to denote corresponding bars in a clear manner, which has been done by giving each symmetric pair the same letter, and following it with l or r in subscript to distinguish the bars on the left from those on the right.

The middle bar, which alone has no duplicate, has been denoted by the single letter z (see Fig. 2).

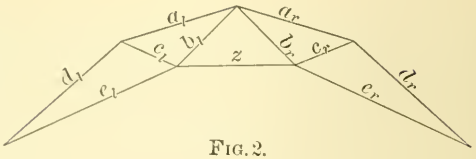


FIG. 2.

The bar z we will treat as the superfluous bar. The stresses S'_{a_r}, S'_{b_r}, etc., due to a tension of unity in z, are as shown in Fig. 3.

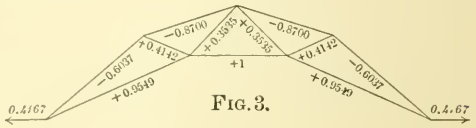


FIG. 3.

We will suppose only the three upper chord joints to be points of loading, neglect dead weight, and consider the various loadings formed by applying the same vertical load P to any one or more of the three points. Denoting the single loadings by L, C or R, according as the load is on the left, center or right joint, we have in single and combined loadings the eight cases, L, C, R, LC, CR, LR, LCR, and "no loading," but of these L and R and LC and CR form symmetrical cases, reducing the loadings requiring separate consideration to six if we confine ourselves to symmetric designs. And only two of these loadings, in addition to the case of "no

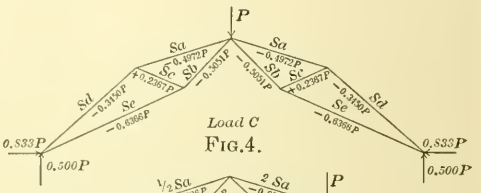


FIG. 4.

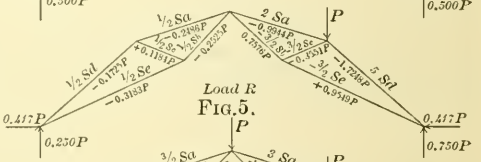


FIG. 5.

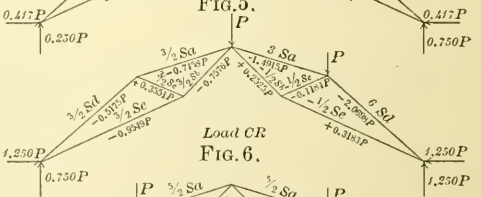


FIG. 6.

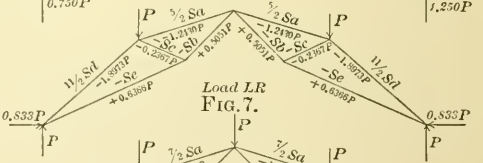


FIG. 7.

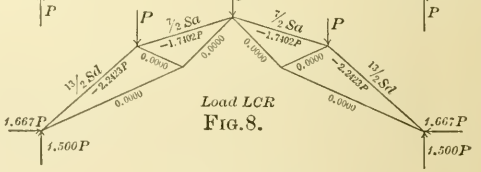


FIG. 8.

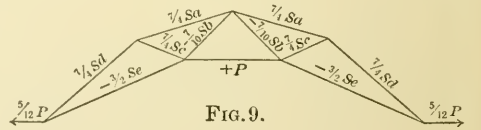


FIG. 9.

loading," require separate calculation, since the others can be obtained through suitably combining these.

The stresses under these various loadings occurring in the statically determined form, that is to say, the three-hinged arch remaining when the middle bar z is omitted, are shown in Figs. 4, 5, 6, 7 and 8.

It will be observed that the stresses of Figs. 6, 7 and 8 are obtained from Figs. 4 and 5 simply by addition.

Denoting the stresses under load C (Fig. 4) by S_a, S_b, S_c, S_d and S_e, simply, we find that the stresses under load R (Fig. 5) may be very conveniently expressed in terms of these, and similarly for all the other loadings (Figs. 6, 7 and 8).

Moreover, comparing Fig. 3 with Fig. 4, we also note relations such that if the tension in z were $+P$, then the stresses, in terms of those in Fig. 4, would be as shown in Fig. 9. Therefore, for convenience in the following we will express all stresses in terms of P, which may be omitted from the equations, but which must be taken into consideration when we come to numerical computations.

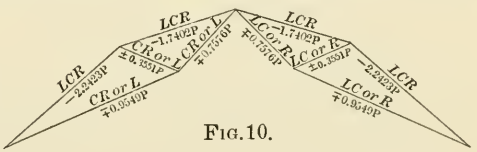


Fig. 10.

Bar	a	b	c	d	e	Total
Volume	$18.166\frac{P}{\sigma}$	$5.357\frac{P}{\sigma}$	$1.912\frac{P}{\sigma}$	$30.174\frac{P}{\sigma}$	$15.866\frac{P}{\sigma}$	$142.830\frac{P}{\sigma}$

Fig. 10 shows the maximum bar stresses of the three-hinged arch together with the loads under which they occur. The bar volumes and total which would correspond to this as a design are also given. We have to find what economy over this can be secured through introducing the middle bar z and making the design indeterminate.

In order to enable us to form a conception of the changes in stress in the indeterminate design, it will be of service to know the changes δz in the middle bar distance under the various loadings in the case of this determinate three-hinged design; for the indeterminate design certainly will have somewhat similar proportions. The shortening of the distance z is $\delta z = \Sigma \frac{S \cdot Sl}{EA}$.

Now the areas are the maximum stresses as given in Fig. 10 divided by the working stress; the quotients being taken posi-

tive. Or we may write $\frac{E}{\sigma}\,\delta\, z = \Sigma\, S\, \frac{S'\, l}{S_{max}}$. In Fig. 11 are the values of $\frac{S'\, l}{S_{max}}$ with the signs of the S', and by aid of these the values of $\frac{E}{\sigma}\,\delta\, z$ were calculated for all the loadings as shown. The range of $\frac{E}{\sigma}\,\delta\, z$ exceeds 60. When the bar z is introduced, that must be so resistent as to reduce the range of $\frac{E}{\sigma}\,\delta\, z$ within 20 since z being 10 ft. long, δz may not vary more than $10\,\frac{\sigma}{E}$ either way from its mean.

Now if we consider the stresses as shown in Figs. 4 to 8, and their modification through introducing a bar z, by aid of the stresses shown in Fig. 3 or Fig. 9, and in the light of this last data as to the character of the variations $\delta\, z$, we see readily that for greatest economy of material in the bars b, c and e the stress in z under L or R must be equal but opposite in sign to that under LC or CR, and similarly under LR equa but of opposite sign to that under C. Therefore the primary stress in z should be equal but of opposite sign to that in z under the full load LCR. This requires that with $\frac{1}{2}\,P$ on each point the bar z shall be free from stress. The greatest stress in z, and therefore the material used in z in making its greatest strain but $\frac{E}{\sigma}$, would also be least for this condition of design, so that it seems probable that this will furnish a, if not the, most economic, indeterminate design. Only the volumes of the bars a and d would be favored by further increasing the resistance of z under the loading LCR, and it does not seem probable that gains there would offset the consequent losses by the bars b, c, e and z. We will therefore seek the particular solution for which the stress in z would be zero with $\frac{1}{2}\,P$ on each point, therefore, for which the stress in z under the various loadings gives the following relations: $S_{z_R} = S_{z_L} = -S_{z_{CR}} = -S_{z_{LC}} = S_5$ say, (Fig. 5 corresponding to loading R), $S_{z_{LR}} = -S_{z_C} = S_7$ say, and $S_{z_{LCR}} = -S_{z_o}$ (the primary stress) $= S_8$ say.

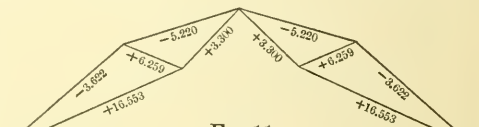


FIG. 11.

Load	C	CR or LR	R or L	LCR	LR	$S_z = P$
$\frac{E}{\sigma}\delta z$	−13.756	+10.328	+24.084	+34.412	+48.168	+52.587

We are now confronted with the question: Under what loadings

will the bars probably have their maximum stresses. It would seem likely that, as in the three-hinged arch, a and d have their greatest stresses under LCR; b, c and e under R and L (CR and LC); and z, judging by the data of Fig. 11, under LR (and C). We will, therefore, first proceed on this basis. This gives the expressions for the section areas A, and the Figs. 4 to 9 enable us to write the stresses as follows:

$$A_a = \frac{-S_{a_{LCR}}}{6} \qquad S_{a_o} = -\tfrac{7}{4}S_a S_8 \qquad S_{a_{LCR}} = +\tfrac{7}{4}S_a\,(2 + S_8)$$

$$A_b = \frac{S_{b_{r_R}}}{6} \qquad S_{b_o} = +\tfrac{7}{10}S_b S_8 \qquad S_{b_{LCR}} = -\tfrac{7}{10}S_b S_8$$

$$A_c = \frac{-S_{c_{r_R}}}{6} \qquad S_{c_o} = -\tfrac{7}{4}S_c S_8 \qquad S_{c_{LCR}} = +\tfrac{7}{4}S_c S_8$$

$$A_d = \frac{-S_{d_{LCR}}}{6} \qquad S_{d_o} = -\tfrac{7}{4}S_d S_8 \qquad S_{d_{LCR}} = +\tfrac{1}{4}S_d(26 + 7S_8)$$

$$A_e = \frac{S_{e_{r_R}}}{6} \qquad S_{e_o} = +\tfrac{3}{2}S_e S_8 \qquad S_{e_{LCR}} = -\tfrac{3}{2}S_e S_8$$

$$A_z = \frac{-S_{z_{LR}}}{6} = \frac{-S_7}{6}$$

$$S_{a_{LR}} = +\tfrac{1}{4}S_a(10 + 7S_7) \qquad S_{a_{r_R}} = +\tfrac{1}{4}S_a(8 + 7S_5)$$

$$S_{b_{LR}} = -\tfrac{1}{10}S_b(10 - 7S_7) \qquad S_{b_{r_R}} = -\tfrac{1}{10}S_b(15 + 7S_5)$$

$$S_{c_{LR}} = -\tfrac{1}{4}S_c(4 - 7S_7) \qquad S_{c_{r_R}} = -\tfrac{1}{4}S_c(6 - 7S_5)$$

$$S_{d_{LR}} = +\tfrac{1}{4}S_d(22 + 7S_7) \qquad S_{d_{r_R}} = +\tfrac{1}{4}S_d(20 + 7S_5)$$

$$S_{e_{LR}} = -\tfrac{1}{2}S_e(2 + 3S_7) \qquad S_{e_{r_R}} = -\tfrac{3}{2}S_e(1 + S_5)$$

$$S_{a_C} = +\tfrac{1}{4}S_a(4 - 7S_7) \qquad S_{a_{l_{CR}}} = +\tfrac{1}{4}S_a(6 - 7S_5)$$

$$S_{b_C} = +\tfrac{1}{10}S_b(10 + 7S_7) \qquad S_{b_{l_{CR}}} = +\tfrac{1}{10}S_b(15 + 7S_5)$$

$$S_{c_C} = +\tfrac{1}{4}S_c(4 - 7S_7) \qquad S_{c_{l_{CR}}} = +\tfrac{1}{4}S_c(6 - 7S_5)$$

$$S_{d_C} = +\tfrac{1}{4}S_d(4 - 7S_7) \qquad S_{d_{l_{CR}}} = +\tfrac{1}{4}S_d(6 - 7S_5)$$

$$S_{e_C} = +\tfrac{1}{2}S_e(2 + 3S_7) \qquad S_{e_{l_{CR}}} = +\tfrac{3}{2}S_e(1 + S_5)$$

$$S_{a_{l_R}} = +\tfrac{1}{4}S_a(2 + 7S_5) \qquad S_{a_{rCR}} = +\tfrac{1}{4}S_a(12 - 7S_5)$$

$$S_{b_{l_R}} = +\tfrac{1}{10}S_b(5 - 7S_5) \qquad S_{b_{rCR}} = -\tfrac{1}{10}S_b(5 - 7S_5)$$

$$S_{c_{l_R}} = +\tfrac{1}{4}S_c(2 + 7S_5) \qquad S_{c_{rCR}} = -\tfrac{1}{4}S_c(2 + 7S_5)$$

$$S_{d_{l_R}} = +\tfrac{1}{4}S_d(2 + 7S_5) \qquad S_{d_{rCR}} = +\tfrac{1}{4}S_d(24 - 7S_5)$$

$$S_{e_{l_R}} = +\tfrac{1}{2}S_e(1 - 3S_5) \qquad S_{e_{rCR}} = -\tfrac{1}{2}S_e(1 - 3S_5)$$

We suppose the coefficient of elasticity to be the same for all bars, therefore, between any two loadings, as $L\,C\,R$ and 0 (no loading), we have equations such as $\Sigma \frac{S'(S_{LCR}-S_0)\,l}{A} = 0$. Writing these for the three pairs of symmetric loadings LCR and 0, R and $C\,R$, and $L\,R$ and C, we have—

$$\frac{-1}{2\sigma}\Sigma\frac{S'(S_{LCR}-S_0)\,l}{A} \equiv \frac{S_a(7+7S_8)\,l_a}{4+2S_8} + \frac{49S_bS_8l_b}{75+35S_5} + \frac{49S_cS_8l_c}{-12+14S_5} + \frac{7S_d\,(13+7S_8)\,l_d}{52+14S_8} + \frac{3S_eS_8l_e}{1+S_5} + \frac{S_8l_z}{S_7} = 0$$

$$\frac{-1}{2\sigma}\Sigma\frac{S'(S_R-S_{CR})l}{A} \equiv \frac{S_a(-2+7S_5)\,l_a}{4+2S_8} + \frac{7S_b\,(5+7S_5)\,l_b}{75+35S_5} + \frac{7S_c(-2+7S_5)\,l_c}{-12+14S_5} + \frac{7S_d(-2+7S_5)\,l_d}{52+14S_8} + \frac{S_e(1+3S_5)\,l_e}{1+S_5} + \frac{S_5l_z}{S_7} = 0$$

$$\frac{-1}{2\sigma}\Sigma\frac{S'(S_{LR}-S_C)\,l}{A} \equiv \frac{S_a(3+7S_7)\,l_a}{4+2S_8} + \frac{7S_b(10+7S_7)\,l_b}{75+35S_5} + \frac{7S_c(-4+7S_7)\,l_c}{-12+14S_5} + \frac{7S_d(9+7S_7)\,l_d}{52+14S_8} + \frac{S_e(2+3S_7)\,l_e}{1+S_5} + l_z = 0$$

Now, if to the first of the above equations we add twice the second and then subtract the third, we obtain the simple relation $S_8 + 2S_5 - S_7 = 0$. This we might otherwise have perceived from the following reasoning: The loading $L\,R$ is obtained by adding to the loading $\frac{1}{2}P$, $\frac{1}{2}P$, $\frac{1}{2}P$ the loading $\frac{1}{2}P$, $-\frac{1}{2}P$, $\frac{1}{2}P$. The loading $L\,C\,R$ is obtained by adding to the loading $\frac{1}{2}P$, $\frac{1}{2}P$, $\frac{1}{2}P$, the loading $\frac{1}{2}P$, $\frac{1}{2}P$, $\frac{1}{2}P$. And half the difference of these additive loads (viz., $\frac{1}{2}\,[(\frac{1}{2}P, -\frac{1}{2}P, \frac{1}{2}P) - (\frac{1}{2}P, \frac{1}{2}P, \frac{1}{2}P)] = (0, -\frac{1}{2}P, 0)$, added to $\frac{1}{2}P$, $\frac{1}{2}P$, $\frac{1}{2}P$ gives the loading $\frac{1}{2}P$, 0, $\frac{1}{2}P$ which would cause the same strain of z as the loading 0, 0, P or loading R. This is a check of the above equations.

Substituting in the second and third of these equations $S_8 + 2S_5$ for S_7 and clearing of fractions in S_8 we get

$$28\,\alpha S^3{}_8 + \begin{bmatrix} (56S_5+160)\,\alpha \\ +14S_a(-2+7S_5)l_a \\ +14S_d(-2+7S_5)l_d \\ +28S_5\,l_z \end{bmatrix} S^2{}_8 + \begin{bmatrix} (320S_5+208)\,\alpha \\ +(28S_5+52)\,S_a(-2+7S_5)l_a \\ +(4S_5+4)\,7S_d(-2+7S_5)l_d \\ +160S_5\,l_z \end{bmatrix} S_8 + \begin{bmatrix} 416S_5\,\alpha \\ +104S_5\,S_a\,(-2+7S_5)\,l_a \\ +56S_5\,S_d\,(-2+7S_5)\,l_d \\ +208S_5\,l_z \end{bmatrix} = 0$$

$$\text{where } \alpha \equiv \frac{7S_b\,(5+7S_5)\,l_b}{75+35S_5} + \frac{7S_c\,(-2+7S_5)\,l_c}{-12+14S_5} + \frac{S_e\,(1+3S_5)\,l_e}{1+S_5},$$

and

$$28\beta S^3{}_8 + \begin{bmatrix} 160\beta \\ +\ 28\gamma \\ +\ 98S_a\,l_a \\ +\ 98S_d\,l_d \end{bmatrix} S^2{}_8 + \begin{bmatrix} -\ 208\beta \\ +\ 160\gamma \\ +\ 14S_a\,(3+14S_5)\,l_a \\ +\ 14S_d\,(9+14S_5)\,l_d \\ +\ 364S_a\,l_a \\ +\ 196S_d\,l_d \end{bmatrix} S_8$$

$$+ \begin{bmatrix} 208\gamma \\ +\ 52S_a\,(3+14S_5)\,l_a \\ +\ 28S_d\,(9+14S_5)\,l_d \end{bmatrix} = 0$$

$$\text{where } \beta \equiv \frac{49S_b\,l_b}{75+35S_5} + \frac{49S_c\,l_c}{-12+14S_5} + \frac{3S_e\,l_e}{1+S_5}.$$

$$\gamma \equiv \frac{7S_b\,(10+14S_5)\,l_b}{75+35S_5} + \frac{7S_c\,(-4+14S_5)\,l_c}{-12+14S_5} + \frac{S_e\,(2+6S_5)\,l_e}{1+S_5}.$$

$$+\ l_z \equiv 2\,\alpha + l_z.$$

The coefficients of S_8 in each of the above cubics in S_8 contain the variable S_5 and otherwise only known quantities. Between the two cubics we could eliminate S_8, obtaining a relation between the coefficients which, cleared of fractions, would yield an equation of about the twentieth degree in S_5. This we could then solve directly. Or, by assuming a series of values of S_5 and determining the corresponding values of S_8 from the cubics, we can plot the curves in part for each of the above equations, and their intersections will furnish the common roots. Adopting this latter method, we find—

if $S_5 = 0$	$S^3{}_8 + 4.798S^2{}_8 + 4.764\,S_8 + 0 = 0$	has root 0
	$S^3{}_8 + 6.885S^2{}_8 + 12.385\,S_8 + 3.975 = 0$	" " -0.406
" $S_5 = -0.1$	$S^3{}_8 + 3.735S^2{}_8 + 1.912\,S_8 + 3.679 = 0$	has root -3.49
	$S^3{}_8 + 6.62\,S^2{}_8 + 10.83\,S_8 + 1.870 = 0$	" " -0.195
" $S_5 = -0.03$	$S^3{}_8 + 4.588S^2{}_8 + 4.139\,S_8 + 0.479 = 0$	has root -0.135
	$S^3{}_8 + 6.820S^2{}_8 + 12.000\,S_8 + 3.441 = 0$	" " -0.354
" $S_5 = -0.05$	$S^3{}_8 + 4.434S^2{}_8 + 3.702\,S_8 + 0.781 = 0$	has root -0.336
	$S^3{}_8 + 6.778S^2{}_8 + 11.743\,S_8 + 3.094 = 0$	" " -0.320

Plotting these, we find an intersection of the curve of the second equation with that of the first at $S_5 = -0.049$ $S_8 = -0.322$, which is

approximately the solution sought. From these values of S_5 and S_8 we get $S_7 = S_8 + 2S_5 = -0.420$.

But this gives us $\begin{matrix} S_{c_{LR}} = -\frac{1}{4} S_c (4 + 7 \times 0.420) = -\frac{1}{4} S_c \times 6.940 \\ S_{c_{r_R}} = -\frac{1}{4} S_c (6 + 7 \times 0.049) = -\frac{1}{4} S_c \times 6.343 \end{matrix}$

or $S^2_{c_{LR}} > S^2_{c_{r_R}}$ which is contrary to our supposition that c has its maximum stress and is designed under the loading R. All other bars have their maximum stresses as supposed. We must therefore substitute $A_c = \frac{-S_{c_{LR}}}{\sigma}$ for its previous value, rewrite our equations and solve again. This affects only one denominator of our equations (page 602). which thus changed become (the last two only being written),

$$\frac{-1}{2\sigma} \Sigma \frac{S'(S_R - S_{cR}) l}{A} \equiv \frac{S_a(-2+7S_5) l_a}{4+2S_8} + \frac{7 S_b (5+7S_5) l_b}{75+35 S_5}$$

$$+ \frac{7 S_c (-2+7S_5) l_c}{-8+14S_7} + \frac{7 S_d (-2+7S_5) l_d}{52+14S_8} + \frac{S_e (1+3S_5) l_e}{1+S_5} + \frac{S_5 l_z}{S_7} = 0$$

$$\frac{-1}{2\sigma} \Sigma \frac{S'(S_{LR} - S_C) l}{A} \equiv \frac{S_a (3+7S_7) l_a}{4+2S_8} + \frac{7 S_b (10+7S_7) l_b}{75+35 S_5} + \frac{7 S_c l_c}{2}$$

$$+ \frac{7 S_d (9+7S_7) l_d}{52+14S_5} + \frac{S_e (2+3S_7) l_e}{1+S_5} + l_z = 0$$

Making the substitution $S_8 + 2S_5$ for S_7 and clearing of fractions in S_8 as before, we obtain two equations, the first a biquadratic, the second a cubic in S_8. We have

$$\left[\begin{matrix} 392\,\alpha' \\ -686\,S_c l_c \end{matrix}\right] S^4{}_8 + \left\{\begin{matrix} (1568 S_5 + 2016)\,\alpha' \\ -49(28 S_5 + 80) S_c l_c \\ +196(-2+7S_5) S_a l_a \\ +196(-2+7S_5) S_d l_d \\ +392 S_5 l_z \end{matrix}\right\} S^3{}_8$$

$$+ \left\{\begin{matrix} (1568 S^2{}_5 + 8512 S_5 + 1632)\,\alpha' \\ -49(160 S_5 + 104) S_c l_c \\ +(784 S_5 + 616)(-2+7S_5) S_a l_a \\ +7(112 S_5 + 40)(-2+7S_5) S_d l_d \\ +(784 S_5 + 2016) S_5 l_z \end{matrix}\right\} S^2{}_8$$

$$+\begin{bmatrix}(8960\,S^2{}_5+9088\,S_5-1664)\,\alpha'\\ -10192\,S_5\,S_c\,l_c\\ +(784\,S^2{}_5+2688\,S_5-416)(-2+7\,S_5)S_a l_a\\ +7(112\,S^2{}_5+192\,S_5-32)(-2+7S_5)S_d\,l_d\\ +(4480\,S_5+1632)\,S_5\,l_z\end{bmatrix}S_8$$

$$+\begin{bmatrix}(11648\,S_5-3328)\,S_5\,\alpha'\\ +(2912\,S_5-832)\,S_5\,(-2+7\,S_5)\,S_a\,l_a\\ +7\,(224\,S_5-64)\,S_5\,(-2+7\,S_5)\,S_d\,l_d\\ +(5824\,S_5-1664)\,S_5\,l_z\end{bmatrix}=0$$

where $\alpha' \equiv \dfrac{7\,S_b\,(5+7\,S_5)\,l_b}{75+35\,S_5}+\dfrac{7\,S_c\,l_c}{2}+\dfrac{S_e\,(1+3\,S_5)\,l_e}{1+S_5}$

and

$$28\beta'\,S^3{}_8+\begin{bmatrix}160\,\beta'\\ +28\,\gamma'\\ +98\,S_a\,l_a\\ +98\,S_d\,l_d\end{bmatrix}S^2{}_8+\begin{bmatrix}208\,\beta'\\ +160\,\gamma'\\ +14(3+14\,S_5)\,S_a\,l_a\\ +14(9+14\,S_5)\,S_d\,l_d\\ +364\,S_a\,l_a\\ +196\,S_d\,l_d\end{bmatrix}S_8$$

$$+\begin{bmatrix}208\,\gamma'\\ +52\,(3+14\,S_5)\,S_a\,l_a\\ +28\,(9+14\,S_5)\,S_d\,l_d\end{bmatrix}=0$$

where $\beta' \equiv \dfrac{49\,S_b\,l_b}{75+35\,S_5}+\dfrac{3\,S_e\,l_e}{1+S_5}$ $\gamma' \equiv 2\,\alpha'+l_z$.

Solving these equations just as in the preceding case we have the following:

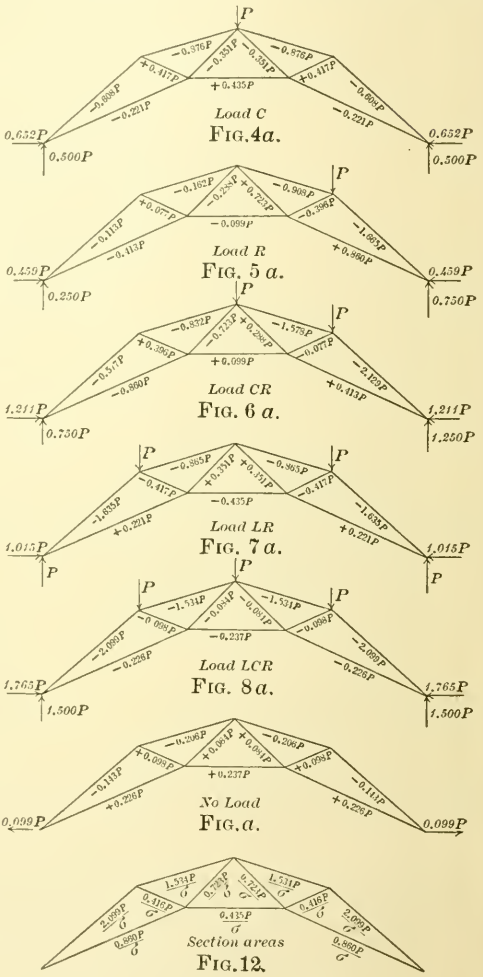
If $S_5=-0.05$ $S^4{}_8+4.394\,S^3{}_8+1.856\,S^2{}_8-1.972\,S_8-0.0309=0$ has root -0.015
$S^3{}_8+6.891\,S^2{}_8+12.151\,S_8+3.278=0$ " " -0.328

" $S_5=-0.10$ $S^4{}_8+3.655\,S^3{}_8+0.8741\,S^2{}_8-1.2567\,S_8-0.3216=0$ has root -0.2558
$S^3{}_8+6.761\,S^2{}_8+11.420\,S_8+2.325=0$ " " -0.2352

" $S_5=-0.098$ $S^4{}_8+3.674\,S^3{}_8+0.9106\,S^2{}_8-1.2996\,S-0.3022=0$ has root -0.2275
$S^3{}_8+6.767\,S^2{}_8+11.450\,S_8+2.363=0$ " " -0.2393

Graphically interpolating, we find approximately $S_5=-0.0990$, $S_8=-0.2373$, which gives $S_7=S_8+2S_5=-0.4353$.

Applying the formulas for the bar stresses written out on page 601 we obtain the series of stresses under the various loadings shown here in figures 4*a* to 8*a* and *a* (compare figures 4 to 8, page 598). But we observe that, contrary to our suppositions, the greatest stresses in the bars *a* and *d* do not occur under *LCR* but under *CR* (and *LC*). Under this latter loading the upper chord stresses are $1\frac{1}{2}$ to $2\frac{1}{2}\%$ greater than under the full loading. The design is not, therefore, admissible, but since it is a close approximation it will be of interest to note the volume of material it takes and compare with that for the three-hinged arch. The section areas are shown in Fig. 12, below which are the bar volumes. In spite of the overstrain of the bars *a* and *d* under the loadings *CR* and *LC*, the economy is but some $4\frac{1}{2}\%$ over the three-hinged arch (see Fig. 10, page 599).



Bar	*a*	*b*	*c*
Volume	$16.01\frac{P}{\sigma}$	$5.11\frac{P}{\sigma}$	$2.25\frac{P}{\sigma}$

d	*e*	*z*	Total
$28.24\frac{P}{\sigma}$	$14.24\frac{P}{\sigma}$	$4.35\frac{P}{\sigma}$	$136.05\frac{P}{\sigma}$

We have now to solve on the supposition that all the bars except *c* and *z* have their greatest stresses under the loadings *CR* and *LC*, so that now

$$A_a = \frac{-S_{a_{r_{CR}}}}{\sigma} = \frac{-\frac{1}{4}S_a^{\frac{1}{2}}(12 - 7S_5)}{\sigma}$$

$$A_a = \frac{-S_{a_{r_{CR}}}}{\sigma} = \frac{-\frac{1}{4}S_a(24 - 7S_5)}{\sigma}.$$

We have, therefore,

$$\frac{-1}{2\sigma}\Sigma\frac{S'(S_R - S_{CR})l}{A} \equiv \frac{7S_a(-2+7S_5)l_a}{24-14S_5} + \frac{7S_b(5+7S_5)l_b}{75+35S_5}$$

$$+\frac{7S_c(2-7S_5)l_c}{8-14S_5} + \frac{7S_d(-2+7S_5)l_d}{48-14S_5} + \frac{S_e(1+3S_5)l_e}{1+S_5} + \frac{S_5 l_z}{S_7} = 0.$$

$$\frac{-1}{2\sigma}\Sigma\frac{S'(S_{LR} - S_C)l}{A} \equiv \frac{7S_a(3+7S_7)l_a}{24-14S_5} + \frac{7S_b(10+7S_7)l_b}{75+35S_5}$$

$$+\frac{7S_c l_c}{2} + \frac{7S_d(9+7S_7)l_d}{48-14S_5} + \frac{S_e(2+3S_7)l_e}{1+S_5} + l_z = 0.$$

These contain only the variables S_5 and S_7. Writing out in terms of S_7 we obtain the equations

$$14\alpha'' S_7^2 + \left[\begin{array}{l} -8\alpha'' \\ -7(2-7S_5)S_c l_c \\ +14S_5 l_z \end{array}\right] S_7 - 8S_5 l_z = 0 \text{ and } \beta'' S_7 + \gamma'' = 0.$$

where

$$\alpha'' \equiv \frac{7S_a(-2+7S_5)l_a}{24-14S_5} + \frac{7S_b(5+7S_5)l_b}{75+35S_5} + \frac{7S_d(-2+7S_5)l_d}{48-14S_5}$$

$$+\frac{S_e(1+3S_5)l_e}{1+S_5}.$$

$$\beta'' \equiv \frac{49S_a l_a}{24-14S_5} + \frac{49S_b l_b}{75+35S_5} + \frac{49S_d l_d}{48-14S_5} + \frac{3S_e l_e}{1+S_5},$$

$$\gamma'' \equiv \frac{21S_a l_a}{24-14S_5} + \frac{70S_b l_b}{75+35S_5} + \frac{7S_c l_c}{2} + \frac{63S_d l_d}{48-14S_5} + \frac{2S_e l_e}{1+S_5} + l_z.$$

Here our equations are much simpler than before, one being linear and the other quadratic in S_7. We could readily eliminate S_7, obtaining an equation of the eleventh degree in S_5, but it will be simpler to proceed as before by assuming values of S_5, computing S_7 and graphically obtaining the common roots.

If $S_5 = -0.100$ $\quad S_7^2 + 0.098495\,S_7 - 0.14068 = 0$ has root -0.4277
$\quad S_7 + 0.4345 = 0$ has root -0.4345.

If $S_5 = -0.101$ $\quad S_7^2 + 0.1087\,S_7 - 0.14349 = 0$ has root -0.4370
$\quad S_7 + 0.4348 = 0$ has root -0.4348.

Graphically interpolating we find

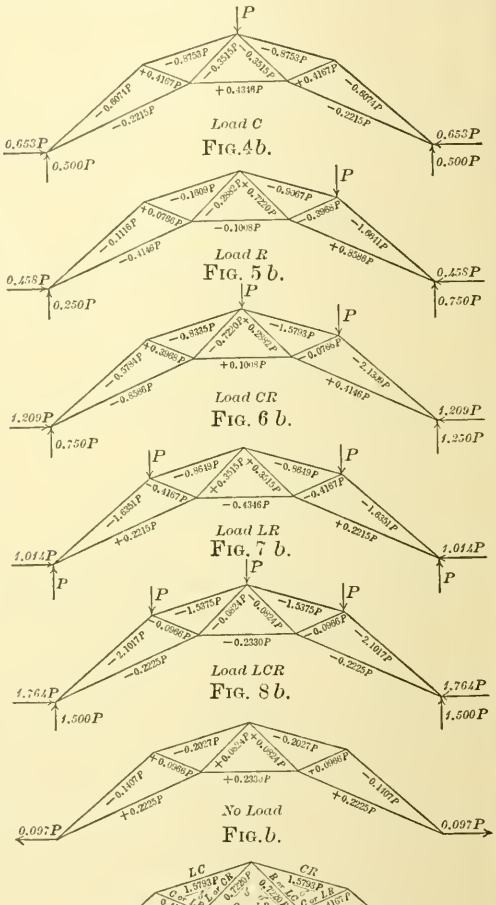
$S_5 = -0.1008.$ $\quad S_7 = -0.4346.$ $\quad S_8 = S_7 - 2S_5 = -0.2330.$

We obtain through the formulas for bar stresses on page 601 the series of stresses under the various loadings shown here in Figs. 4*b* to 8*b* and *b*.

This time the greatest stresses all occur under the loads as assumed, and the solution here obtained is therefore that sought.

The section areas are as given in Fig. 13, together with the loadings under which they are designed. And below are given the volumes of the bars and the total volume (compare Fig. 10, page 599). The economy thus obtained over the three-hinged arch proves to be but $3\frac{1}{2}$ per cent.

We may test the work by computing the shortening of the line z from its length when the bars a, b, c, d and e all have zero strain. We have $\frac{E}{\sigma}\,\delta\, z = \Sigma \frac{S'l}{S_{max}} S$ (if we write S_{max} always +). The values of $\frac{S'l}{S_{max}}$ being as shown in Fig. 14, we obtain, by aid of the stresses given in Figs. 4*b* to 8*b* and *b*, the values of $\frac{E}{\sigma}\,\delta\, z$ shown above.



Bar	a	b	c	d
Volume	$16.49\frac{P}{\sigma}$	$5.11\frac{P}{\sigma}$	$2.24\frac{P}{\sigma}$	$28.67\frac{P}{\sigma}$

Bar	e	z	Total
Volume	$14.21\frac{P}{\sigma}$	$4.34\frac{P}{\sigma}$	$137.78\frac{P}{\sigma}$

Subtracting from the others the value for $\frac{1}{2}(LCR)$ when the bar z has no strain, we obtain $\frac{E}{\sigma}$ times the actual strain of z. This should be ± 10 (more accurately $\pm l_z$ which is somewhat less than 10 ft.), for loads LR and C, respectively, $\pm \frac{1008}{4346} \times 10 = \pm 2.319$ for R and CR, respectively, and $\pm \frac{2330}{4346} \times 10 = \pm 5.361$ for LCR and "no load," respectively. Even these, which are a trifle too large owing to the value of l_z used, check within $\frac{2}{10}\%$, or within the probable error from keeping but four significant figures in the calculations.

The normal or unstrained length of the bar z is its length under the loading which makes its stress zero, that is $\frac{1}{2}P$ on each panel point. This length is $(10 - 18.556 \frac{\sigma}{E}$ ft.). Taking $\sigma = 10\,000$ lbs. per square inch, and $E = 30\,000\,000$ lbs. per square inch, this correction is 0.00618 ft., the corrected length of z being 9.99382 ft. It is seen that an error of a ten-thousandth of a foot would sensibly change the strain, and the same is true of the other bars. To ensure the distribution of the stresses as here calculated, within, say, 1%, the bar lengths should all be calculated to five decimal places and single errors in the finished structure should not much exceed a unit in the last decimal place. That is, the bar lengths should be true to about one ten-thousandth of an inch; certainly closer than to one-thousandth of an inch. And we must further suppose no comparable yielding of joints or abutments.

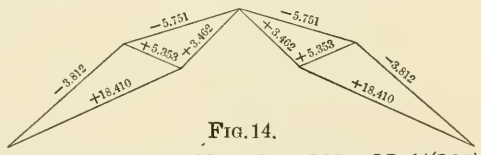


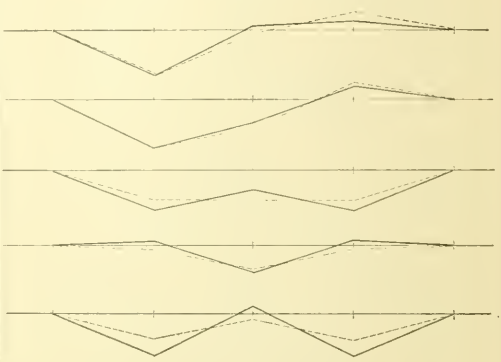
FIG. 14.

Load	C	*No Load*	CR	R	LCR	LR	$\frac{1}{2}(LCR)$
$\frac{E}{\sigma}\delta z$	+8.572	+13.198	+16.242	+20.871	+23.912	+28.540	+18.556
	−9.984	−5.358	−2.314	+2.315	+5.356	+9.984	0.000

The relative stiffness of the indeterminate design just obtained as compared with that of the determinate (three-hinged) design is a matter of much interest, since it is frequently claimed that the two-hinged arch is stiffer than the three-hinged arch. If by S' we indicate the stresses in the three-hinged arch due to a vertical load of unity on any joint and by S the actual stresses in the bars under any given loading, in either design, then $\Sigma \frac{S'Sl}{EA}$ will give the sinking of the joint in question under the given loading. The values of S' for load

unity on the middle joint or on the right upper chord joint we readily obtain from Figs. 4 and 5. the values of S we obtain from Figs. 4 to 8 and Figs. $4b$ to $8b$ and b, and the values of A from Fig. 10 and Fig. 13. We note that for the indeterminate design "no load" also gives deflections, $-3.72\ \frac{\sigma}{E}$ at the middle joint and $+6.19\ \frac{\sigma}{E}$ at the side upper chord joints. The deflections, corrected as above for the indeterminate design and without the factor $\frac{\sigma}{E}$, are as follows:

Load	Joint	Deflection; Three-hinged arch.	Deflection; Two-hinged arch.
L	L	+44.96	+44.64
	C	−2.63	+2.52
	R	−8.84	−18.40
LC	L	+47.33	+47.18
	C	+22.29	+26.12
	R	−11.47	−15.86
LCR	L	+38.50	+28.79
	C	+19.66	+23.64
	R	+38.50	+28.79
C	L	−2.63	+2.54
	C	+24.92	+23.59
	R	−2.63	+2.54
LR	L	+41.13	+26.24
	C	−5.25	+5.05
	R	+41.13	+26.24



We note that the deflections differ in character in the two cases. The greatest deflections do not occur under the same loadings. The greatest side joint deflection of the three-hinged arch is some 6% greater than that of the two-hinged arch, but the greatest center joint deflection of the three-hinged arch is some 5% less than that of the two-hinged arch. The center deflections are little more than half the quarter deflections in both cases. Practically one arch is about as stiff as the other. Certainly the existence of the middle bar in the two-hinged arch does not appear to have given it a pronounced advantage.

Is the indeterminate solution which we have found one of greatest economy of material? It at least is a solution at a point of discontinuity in the variation of the material used. That is, under our condition of equal maximum stress, if the compression in z under R were slightly less, the tension in z under $C\ R$ would be slightly greater, A_b and A_r would be determined under the loading R, and A_c and A_z would be determined under the loading C. But if the compression

in z under R were slightly greater, the tension in z under $C\,R$ would be slightly less, A_b and A_e would be determined under the loading $C\,R$ and A_c and A_z would be determined under the loading $L\,R$. For, consider the three equations between R and $C\,R$, $L\,R$ and C, and R and $L\,R$ in the former case. Put $S_{z_R} = S_5 + \Delta_5$, $S_{z_{CR}} = -S_5 + \Delta_6$, $S_{z_{LR}} = S_7 + \Delta_7$ and $S_{z_C} = -S_7 + \Delta_4$, where Δ_5 is $+$. Then

$$
\begin{aligned}
&-\frac{1}{\sigma}\Sigma\frac{S'(S_R - S_{CR})\,l}{A}\\
&\equiv\frac{7\,S_a(-4+14\,S_5+7\Delta_5-7\Delta_6)\,l_a}{24-14\,S_5+14\Delta_6}\\
&+\frac{7\,S_b(10+14\,S_5+7\Delta_5-7\Delta_6)\,l_b}{75+35\,S_5+35\Delta_5}\\
&+\frac{7\,S_c(4-14\,S_5-7\Delta_5+7\Delta_6)\,l_c}{8-14\,S_7+14\Delta_4}\\
&+\frac{7\,S_d(-4+14\,S_5+7\Delta_5-7\Delta_6)\,l_d}{48-14\,S_5+14\Delta_6}\\
&+\frac{S_e(2+6\,S_5+3\Delta_5-3\Delta_6)\,l_e}{1+S_5+\Delta_5}\\
&+\frac{(2\,S_5+\Delta_5-\Delta_6)\,l_z}{S_7-\Delta_4}\\
&=0
\end{aligned}
\qquad
\begin{aligned}
&-\frac{1}{\sigma}\Sigma\frac{S'(S_{LR} - S_C)\,l}{A}\\
&\equiv\frac{7\,S_a(6+14\,S_7+7\Delta_7-7\Delta_4)\,l_a}{24-14\,S_5+14\Delta_6}\\
&+\frac{7\,S_b(20+14\,S_7+7\Delta_7-7\Delta_4)\,l_b}{75+35\,S_5+35\Delta_5}\\
&+\frac{7\,S_c(8-14\,S_7-7\,\Delta_7+7\,\Delta_4)\,l_c}{8-14\,S_7+14\Delta_4}\\
&+\frac{7\,S_d(18+14\,S_7+7\Delta_7-7\Delta_4)\,l_d}{48-14\,S_5+14\Delta_6}\\
&+\frac{S_e(4+6\,S_7+3\,\Delta_7-3\,\Delta_4)\,l_e}{1+S_5+\Delta_5}\\
&+\frac{(2S_7+\Delta_7-\Delta_4)\,l_z}{S_7-\Delta_4}\\
&=0
\end{aligned}
$$

$$
\begin{aligned}
&-\frac{1}{\sigma}\Sigma\frac{S'(S_{LR} - S_R)\,l}{A}\\
&\equiv\frac{7\,S_a(5+7\,S_7-7\,S_5+7\Delta_7-7\,\Delta_5)\,l_a}{24-14\,S_5+14\Delta_6}\\
&+\frac{7\,S_b(5+7\,S_7-7\,S_5+7\,\Delta_7-7\,\Delta_5)\,l_b}{75+35\,S_5+35\,\Delta_5}\\
&+\frac{7\,S_c(2-7\,S_7+7\,S_5-7\Delta_7+7\,\Delta_5)\,l_c}{8-14\,S_7+14\,\Delta_4}\\
&+\frac{7\,S_d(11+7\,S_7-7\,S_5+7\,\Delta_7-7\,\Delta_5)l_d}{48-14\,S_5+14\,\Delta_6}\\
&+\frac{S_e(1+3\,S_7-3\,S_5+3\,\Delta_7-3\,\Delta_5)\,l_e}{1+S_5+\Delta_5}\\
&+\frac{(S_7-S_5+\Delta_7-\Delta_5)\,l_z}{S_7-\Delta_4}\\
&=0
\end{aligned}
$$

Remembering that the Δ are all very small, and noting the two equations on page 607 and also the equation formed by their difference, the equations just written may be reduced to:

$$-\frac{1}{\sigma}\Sigma\frac{S'(S_R-S_{CR})\,l}{A}$$

$$=\frac{7S_a(7\Delta_5-7\Delta_6+\frac{4-14S_5}{24-14S_5}\times 14\Delta_6)\,l_a}{24-14S_5}$$

$$+\frac{7S_b(7\Delta_5-7\Delta_6-\frac{10+14S_5}{75+35S_5}\times 35\Delta_5)\,l_b}{75+35S_5}$$

$$+\frac{7S_c(-7\Delta_5+7\Delta_6-\frac{4-14S_5}{8-14S_7}\times 14\Delta_4)\,l_c}{8-14S_7}$$

$$+\frac{7S_d(7\Delta_5-7\Delta_6+\frac{4-14S_5}{48-14S_5}\times 14\Delta_6)\,l_d}{48-14S_5}$$

$$+\frac{S_e(3\Delta_5-3\Delta_6-\frac{2+6S_5}{1+S_5}\times\Delta_5)\,l_e}{1+S_5}$$

$$+\frac{(\Delta_5-\Delta_6+\frac{2S_5}{S_7}\times\Delta_4)\,l_z}{S_7}$$

$$=0$$

$$-\frac{1}{\sigma}\Sigma\frac{S'(S_{LR}-S_C)\,l}{A}$$

$$=\frac{7S_a(7\Delta_7-7\Delta_4-\frac{6+14S_7}{24-14S_5}\times 14\Delta_6)\,l_a}{24-14S_5}$$

$$+\frac{7S_b(7\Delta_7-7\Delta_4-\frac{20+14S_7}{75+35S_5}\times 35\Delta_5)\,l_b}{75+35S_5}$$

$$+\frac{7S_c(-7\Delta_7+7\Delta_4-14\Delta_4)\,l_c}{8-14S_7}$$

$$+\frac{7S_d(7\Delta_7-7\Delta_4-\frac{18+14S_7}{48-14S_5}\times 14\Delta_6)\,l_d}{48-14S_5}$$

$$+\frac{S_e(3\Delta_7-3\Delta_4-\frac{4+6S_7}{1+S_5}\times\Delta_5)\,l_e}{1+S_5}$$

$$+\frac{(\Delta_7-\Delta_4+2\Delta_4)\,l_z}{S_7}$$

$$=0$$

$$-\frac{1}{\sigma}\Sigma\frac{S'\,(S_{LR}-S_R)\,l}{A}$$

$$=\frac{7S_a(7\Delta_7-7\Delta_5-\frac{5+7S_7-7S_5}{24-14S_5}\times 14\Delta_6)\,l_a}{24-14S_5}$$

$$+\frac{7S_b\,(7\Delta_7-7\Delta_5-\frac{5+7S_7-7S_5}{75+35S_5}\times 35\Delta_5)\,l_b}{75+35S_5}$$

$$+\frac{7S_c(-7\Delta_7+7\Delta_5-\frac{2-7S_7+7S_5}{8-14S_7}\times 14\Delta_4)\,l_c}{8-14S_7}$$

$$+\frac{7S_d(7\Delta_7-7\Delta_5-\frac{11+7S_7-7S_5}{48-14S_5}\times 14\Delta_6)\,l_d}{48-14S_5}$$

$$+\frac{S_e(3\Delta_7-3\Delta_5-\frac{1+3S_7-3S_5}{1+S_5}\times\Delta_5)\,l_e}{1+S_5}$$

$$+\frac{(\Delta_7-\Delta_5+\frac{S_7-S_5}{S_7}\times\Delta_4)\,l_z}{S_7}$$

$$=0.$$

These are three linear equations between the four quantities Δ_4, Δ_5, Δ_6 and Δ_7.

Putting in the numerical values of S_a, S_b, S_c, S_d, S_e, l_a, l_b, l_c, l_d, l_e, l_z, and of S_5 and S_7, these equations may be reduced to the numerical equations:

$$\left.\begin{aligned} &\Delta_4 + 4.26\Delta_5 - 5.29\Delta_6 = 0 \\ &\Delta_4 + 0.82\Delta_5 + 0.09\Delta_6 - 3.21\Delta_7 = 0 \\ &\Delta_4 - 3.92\Delta_5 - 0.18\Delta_6 + 3.90\Delta_7 = 0 \end{aligned}\right\} \text{ which give } \left\{\begin{aligned} \Delta_5 &= +1.4\Delta_7 \\ \Delta_6 &= +1.5\Delta_7 \\ \Delta_4 &= +1.9\Delta_7 \end{aligned}\right.$$

Thus the Δ are all + as stated. But this change then increases all of the designing stresses $S_{a_{r_{CR}}}$, $S_{b_{r_R}}$, S_{c_C}, $S_{d_{r_{CR}}}$, $S_{e_{r_R}}$, and S_{z_C}, and therefore increases the material employed.

Now, consider the similar equations between the loadings R and CR, LR and C, and R and LR in the second case, when Δ_5 is —, and a, b, d and e are designed under CR, and c and z under LR. We may at once write them down in the second form, the change being simply replacing Δ_5 by $-\Delta_6$ and Δ_4 by $-\Delta_7$ where, in the brackets, these have a fraction for a coefficient. We have:

$$-\frac{1}{\sigma}\Sigma\frac{S'(S_R - S_{CR})l}{A}$$

$$= \frac{7S_a(7\Delta_5 - 7\Delta_6 + \frac{4-14S_5}{24-14S_5}\times 14\Delta_6)\,l_a}{24-14S_5}$$

$$+ \frac{7S_b(7\Delta_5 - 7\Delta_6 + \frac{10+14S_5}{75+35S_5}\times 35\Delta_6)\,l_b}{75+35S_5}$$

$$+ \frac{7S_c(-7\Delta_5 + 7\Delta_6 + \frac{4-14S_5}{8-14S_7}\times 14\Delta_7)\,l_c}{8-14S_7}$$

$$+ \frac{7S_d(7\Delta_5 - 7\Delta_6 + \frac{4-14S_5}{48-14S_5}\times 14\Delta_6)\,l_d}{48-14S_5}$$

$$+ \frac{S_e(3\Delta_5 - 3\Delta_6 + \frac{2+6S_5}{1+S_5} + \Delta_6)\,l_e}{1+S_5}$$

$$+ \frac{(\Delta_5 - \Delta_6 - \frac{2S_5}{S_7} + \Delta_7)\,l_z}{S_7}$$

$$= 0$$

$$-\frac{1}{\sigma}\Sigma\frac{S'(S_{LR} - S_C)\,l}{A}$$

$$= \frac{7S_a(7\Delta_7 - 7\Delta_4 - \frac{6+14S_7}{24-14S_5}\times 14\Delta_6)\,l_a}{24-14S_5}$$

$$+ \frac{7S_b(7\Delta_7 - 7\Delta_4 + \frac{20+14S_7}{75+35S_5}\times 35\Delta_6)\,l_b}{75+35S_5}$$

$$+ \frac{7S_c(-7\Delta_7 + 7\Delta_4 + 14\Delta_7)\,l_c}{8-14S_7}$$

$$+ \frac{7S_d(7\Delta_7 - 7\Delta_4 - \frac{18+14S_7}{48-14S_5}\times 14\Delta_6)\,l_d}{48-14S_5}$$

$$+ \frac{S_e(3\Delta_7 - 3\Delta_4 + \frac{4+6S_7}{1+S_5}\times\Delta_6)\,l_e}{1+S_5}$$

$$+ \frac{(\Delta_7 - \Delta_4 - 2\Delta_7)\,l_z}{S_7}$$

$$= 0$$

$$
\begin{aligned}
&-\frac{1}{\sigma}\quad\frac{S'(S_{LR}-S_R)\,l}{A}\\
&=\frac{7S_a\,(7\Delta_7-7\Delta_5-\dfrac{5+7S_7-7S_5}{24-14S_5}\times 14\Delta_6)\,l_a}{24-14S_5}\\
&+\frac{7S_b\,(7\Delta_7-7\Delta_5+\dfrac{5+7S_7-7S_5}{75+35S_5}\times 35\Delta_6)\,l_b}{75+35S_5}\\
&+\frac{7S_c\,(-7\Delta_7+7\Delta_5+\dfrac{2-7S_7+7S_5}{8-14S_7}\times 14\Delta_7)\,l_c}{8-14S_7}\\
&+\frac{7S_d\,(7\Delta_7-7\Delta_5-\dfrac{11+7S_7-7S_5}{48-14S_5}\times 14\Delta_6)\,l_d}{48-14S_5}\\
&+\frac{S_e(3\Delta_7-3\Delta_5+\dfrac{1+3S_7-3S_5}{1+S_5}\times\Delta_6)\,l_e}{1+S_5}\\
&+\frac{(\Delta_7-\Delta_5-\dfrac{S_7-S_5}{S_7}\times\Delta_7)\,l_z}{S_7}\\
&=0
\end{aligned}
$$

These give three linear equations between the four quantities Δ_4, Δ_5, Δ_6 and Δ_7, which may be reduced to the numerical equations:

$$
\left.\begin{aligned}
\Delta_5-0.687\Delta_6-0.177\Delta_7&=0\\
\Delta_4-0.230\Delta_6-0.313\Delta_7&=0\\
\Delta_5+0.041\Delta_6-0.744\Delta_7&=0
\end{aligned}\right\}\ \text{which give....}
$$

$$
\cdots\cdots\left\{\begin{aligned}
\Delta_6&=+0.78\Delta_7 \qquad \Delta_7=\frac{+1}{0.78}\quad \Delta_6=+1.28\Delta_6\\
\Delta_5&=+0.71\Delta_7\\
\Delta_4&=+0.49\Delta_7
\end{aligned}\right.
$$

Thus the Δ are all of the same sign, or negative, and a, b, d and e have their greatest stresses under CR, and c and z theirs under LR, as stated. The changes in volume due to a change Δ_6 (increased compression in z under CR) are:

Volume of b increases $S'_b l_b \Delta_6 \frac{1}{\sigma} = 2.50 \Delta_6 \frac{1}{\sigma}$

" " e " $S'_e l_e \Delta_6 \frac{1}{\sigma} = 15.80 \Delta_6 \frac{1}{\sigma}$

" " c " $S'_c l_c \Delta_7 \frac{1}{\sigma} = 2.85 \Delta_6 \frac{1}{\sigma}$

$21.15 \Delta_6 \frac{1}{\sigma}$

2

" " $2(b + c + e)$ increases $= 42.30 \Delta_6 \frac{1}{\sigma}$

" " z " $l_z \Delta_7 \frac{1}{\sigma} = 12.80 \Delta_6 \frac{1}{\sigma}$

" " $2(b + c + e) + z$ increases $55.10 \Delta_6 \frac{1}{\sigma}$

Volume of a diminishes $S'_a l_a \Delta_6 \frac{1}{\sigma} = 9.08 \Delta_6 \frac{1}{\sigma}$

" " d " $S'_a l_a \Delta_6 \frac{1}{\sigma} = 8.12 \Delta_6 \frac{1}{\sigma}$

$17.20 \Delta_6 \frac{1}{\sigma}$

2

" " $2(a + d)$ diminishes $= 34.40 \Delta_6 \frac{1}{\sigma}$

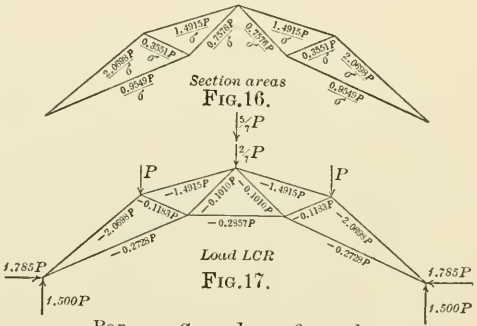
Thus this variation also increases the material used, the saving of material on the bars a and d being considerably more than offset by the increased material in the bars b, c, e and z. Thus the indeterminate solution we have found is at a point of discontinuity which is a point of greatest economy. Now it is readily shown that a larger primary stress in z is always less economic, and simple investigation shows that the smaller the primary tension in z and therefore the greater its greatest compression, the less saved on the bars a and d compared with that expended on the bars b, c, e and z. This solution is therefore the most economic symmetric solution; and as investigation shows that unsymmetric solutions are less economic than the symmetric, we may conclude that the design obtained is the most economic solution of the given two-hinged arch for the given loadings, on the basis of a uniform greatest mean stress in designing.

Now, the economic superiority of this most economic indeterminate design over the simple determinate design (the three-hinged arch), is but some $3\frac{1}{2}\%$ (see page 608). Such a small saving, if all other things were equal, would hardly be worth the labor of seeking; but we recognize that, aside from the difficulty of determining the design, other things are not at all equal, for, to actually realize the stress distribution calculated, so accurately as to really preserve the $3\frac{1}{2}\%$ advantage, extremely accurate and therefore expensive work would be necessary, and also very rigid and therefore expensive abutments. Then, once constructed, no set may occur under use, or conditions would not be maintained as supposed; and last, but not least, changes in temperature are not allowable since they would greatly change the stresses. Thus, with $E = 30\,000\,000$ lbs. per square inch and $\sigma = 10\,000$ lbs. per square inch, a change of 0.064 ft. in the span would cause a stress of P in the bar z, and the corresponding stresses in the remaining bars (see Fig. 3 or Fig. 9). With a coefficient of expansion of $\frac{6\frac{1}{2}}{1\,000\,000}$ per degree Fahrenheit, 100° Fahr. would correspond to a change in span of $\frac{6\frac{1}{2} \times 100 \times 40}{1\,000\,000} = 0.4 \times 0.065$ ft., or $\frac{1}{10}$ P stress in z, nearly. This result is shown in Fig. 15. And while 100° Fahr. change from the mean (that is 200° Fahr. range of temperature), may be excessive, even if we take only half that or 50° Fahr. change (100° Fahr. range), we get temperature stresses of half those shown in Fig. 15 which, by comparison with those in Fig. 13, we see are some 45% of the greatest stress in z, 22% of that in e, 20% of that in c, 11% of that in a, 10% of that in b, and 6% of that in d. That these stresses, absent from the three-hinged arch, turn the balance completely in its favor, many times offsetting the $3\frac{1}{2}\%$ economy in the two-hinged arch found by neglecting temperature changes, is self-evident to any impartial student of the subject.

FIG. 15.
Temperature stresses for +100° Fahr.

Finally, we may note that even the $3\frac{1}{2}\%$ theoretic advantage of the two-hinged arch can be exceeded without the use of indeterminate construction through the use of what has elsewhere been termed "multiple construction." Thus we have the three-hinged arch design

of Fig. 16, capable of supporting all the loadings except $L\,C\,R$ with stress σ or less (see Figs. 4, 5, 6 and 7). For the load $L\,C\,R$ we introduce a bar z of area $\frac{2}{7}\frac{P}{\sigma}$, give the junction of the bars a_l and a_r freedom to move vertically with respect to the junction of the bars b_l and b_r, and load the junction of $a_l a_r$ with $\frac{5}{7}P$, and the junction of $b_l b_r$ with the remainder or $\frac{2}{7}P$. The resulting stresses in this framework, which is statically determined, are shown in Fig. 17. We see that these stresses are equal to or less than the designing stresses shown in Fig. 16. The construction permitting of this might be any such as would ordinarily leave z unstressed, but, as by the insertion of a wedge, cause it to take compression when desired, while the joint of $a_l a_r$ could at the same time be freed from the joint of $b_l b_r$, say by removing another wedge. But it suffices to note that the construction is possible; with the mechanical details we are not here concerned.



Bar	a	b	c	d
Volume	$15.57\frac{P}{\sigma}$	$5.36\frac{P}{\sigma}$	$1.91\frac{P}{\sigma}$	$27.85\frac{P}{\sigma}$
Bar	e	z	Total	
Volume	$15.81\frac{P}{\sigma}$	$2.86\frac{P}{\sigma}$	$135.84\frac{P}{\sigma}$	

The volume of material here, as shown under Fig. 17, is some $1\frac{1}{2}\%$ less than that required by the most economic indeterminate design. Very possibly other multiple determinate designs might be found with still somewhat more economy of material. But with these we need not concern ourselves, the essential point being simply that, as has been stated previously, there will be some statically determined method of solution more economic than any indeterminate solution, even on the theoretic conditions most favorable to the latter. It is not intended to intimate, however, that these multiple determinate solutions are in general desirable substitutes for the simpler but less economic single determinate solutions. The multiple solutions present more or less considerable mechanical difficulties in their adaptation to practical use, and while these difficulties are not

insuperable, in fact, perhaps, are less considerable than the exact construction of an indeterminate framework, they would probably prevent their use where the saving was not considerable. In a case like the present, where the saving would be but some 5%, it is hardly likely that they would be used.

This simple example of the indeterminate arch, although valueless as a design because of the neglect of post formulas, as a comparative study, is affected little by such considerations, and illustrates very well certain points in economic design. While, theoretically, the indeterminate framework may give a more economic design for several loadings, than does any single determinate framework of included figure, the amount of such theoretic advantage is considerable only in most extraordinary cases; and there it would probably pay to use a multiple determinate design which would be still more economic. In all cases where the figure of the simple determinate design closely approximates to the figure of the indeterminate design the theoretic advantage possible through using the latter will certainly be insignificant. Actually, owing to temperature stresses, etc., that advantage will not exist at all. Again, as the structure becomes larger, and the variations of stress due to variable loadings affect smaller and smaller parts of the structure to a sensible degree, the possible economy of indeterminate construction proportionally diminishes. It may safely be assumed that in actual structures (as arches and suspension bridges), a theoretic advantage of even 5% cannot exist in the most economic indeterminate design over the most economic determinate design of included figure, even when temperature stresses are neglected. It is doubtful if ordinarily 1% economy could be found; and temperature stresses alone would offset such advantages several times over in all cases. This illustration, then, definitely confirms the view that the use of indeterminate frameworks is certainly not advantageous from an economic standpoint, nor apparently from any other standpoint.

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PAPERS AND DISCUSSIONS.

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RIVER HYDRAULICS.

By JAMES A. SEDDON, M. Am. Soc. C. E.

TO BE PRESENTED DECEMBER 6TH, 1899.

The following is a brief outline of the methods and results of a systematic study of data taken on the Mississippi River and its tributaries. The system of rivers, the principal drainage divisions and a number of locations to which it will be necessary to refer, are shown on the map (Fig. 1).

As the writer is satisfied that this study has now reached such a point that it will enable river engineers to see clearly what observations should be made and what conclusions might be drawn from them, thereby being the means of saving much time and money, he has ventured to give it as it is, notwithstanding the fact that it contains a number of questions as yet unanswered.

As an introduction, it may be noted that, in working in an established field of science, the relation between phenomena and their fundamental principles is all that need be considered, but it is somewhat different where the phenomena are simply a mass of facts and no fundamental principles are given. In such cases the first step is to find the relations between the given facts; the fundamental principles in which these relations are included will come later.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in *Transactions*.

This is the manner in which this study was made, and its results here presented. The data are subdivided into the different classes in which these simple relations of fact were first sought, and an analysis is then given in which these again are all included in a fundamental relation.

In presenting these relations of fact, each of which has been dug slowly out of great masses of data on various rivers, in some cases covering periods of half a century, the writer has in each case used as an illustration a single instance of the relation in its simplest form, and has noted briefly the variations from this simple form which may occur, and their causes. Thus, the relation between the gauge readings along the river for its own floods is one of these primary relations, while the effects of a flood out of a tributary between any two of the gauges, or the cutting through a bend and the total change of channel near one of them, are considered as variations.

Anyone may test the relation easily, by referring to the published data in the cases given; but, after that, it should be understood that the relation is not to be discredited by selecting other periods, in which it is not so simple, without at the same time considering their variations. The whole conclusion lies often in hundreds and sometimes even in thousands of plates of data, which, of course, cannot be given here, and, so far, the writer must be trusted. The test comes in the application of the final analysis, and, if this bears the test, it will not be necessary for anyone else to go through again the laborious process which he has had to follow.

Slope and Section Data.

It is not the purpose here to attempt to add anything to the data or the theory of velocity-slope relations; nor, indeed, important as they are to the engineer who lays a pipe or builds a conduit, to consider them even directly. Where man makes the form and sets the slope, the resulting flow is the one thing that he wants to know, but where the flow makes its own form, and that a very irregular one, and so distributes its slope that it may be almost all concentrated at certain points at one stage, with little or none there at a different stage, there is not much use in trying to express it as $v = c\sqrt{r\,s}$, for r and s are even more difficult to measure than v.

Of course the fall is determined quite accurately over long reaches,

PACIFIC OCEAN
MEXICO
CALIFORNIA
NEVADA
OREGON
WASHINGTON
IDAHO
UTAH
ARIZONA
COLORADO
NEW MEXICO
WYOMING
MONTANA
DOMINION
TEXAS
OKLAHOMA
KANSAS
NEBRASKA
S. DAKOTA
N. DAKOTA
MINNESOTA
IOWA
MISSOURI
ARKANSAS
LOUISIANA
MISSISSIPPI
ILLINOIS
WISCONSIN
MICHIGAN
INDIANA
OHIO
KENTUCKY
TENNESSEE
ALABAMA
GEORGIA
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S. CAROLINA
N. CAROLINA
VIRGINIA
PENNSYLVANIA
NEW YORK
VERMONT
N. HAMP.
MASS.
CONN.
MAINE
CANADA
GULF OF MEXICO
ATLANTIC OCEAN
L. Michigan
L. Superior
L. Erie
L. Ontario
St. Lawrence
Rio Grande
Missouri
Columbia
Snake
Colorado
Arkansas
Red River
Ohio
Tennessee
Cumberland
Yellowstone

Fig. 1

but to determine the values of the mean velocity and the mean hydraulic radius which correspond to it may require a year's observations and survey, while if these are taken in the selected section the slope there may be found to be even reversed, or the water surface may be found to slope in the opposite direction to the flow. It is not at all impossible that, taking a hundred miles of river a mile wide, this formula may in certain cases express the general relations quite as well as it does in 100 ft. of pipe 1 ft. in diameter, but such a relation in the river is wholly useless; while it is equally possible that, if the relation were sought in a 1-ft. length of the pipe, it would be found altogether as erratic as it is for the selected section of the river.

True, there is a difference between the foot of pipe and the mile of river; the theoretical equivalent is probably given more nearly by a relation between resistance and momentum, or the lengths of a fall equal to the given velocities; but, in addition to this, the give and take of stored energy in the river is to be considered. The Lower Mississippi at its bank-full stage has at least a range of from 10 000 to 30 000 foot-tons in the energy of its flow, and in the play of such a balance wheel local slope-velocity relations very naturally are obliterated.

Not only, however, are no such local relations to be looked for in rivers, but also, even if such a general relation should exist at one stage, such a general relation at all stages by no means follows. In many cases the alternations of pool and bar at low water do not differ greatly from a chain of small reservoirs, each emptying through a weir into that below it; and in that case the level and the discharge of each are not affected in the least by the vertical distance which one may be put below the other. True, they may wear under the fall, unless the pool is deep enough and the material is stable enough, but the only slope relation there is this limiting relation with the size of the pools and the character of the bottom.

Where the bed is unyielding and such a condition has simply been formed in the slow process of geological periods, the discharge from pool to pool is no function of slope whatever, but a function of stage. Were the connections plain weirs of given forms, it might be calculated for any level, but as it is more in the nature of a chain of drowned weirs of very irregular forms, the only way to get it is to measure it. In either case, however, it is wholly independent of the different slopes

between the rising and falling river, and it is determined solely by the pool level.

But the concentration of slope between the pools at low water may be entirely reversed at high water, and the narrow sections there control the flow into the series of shallower but broader levels between them; or in this case, the type of control has passed from that of a horizontal weir to that of a vertical orifice, but one is just as arbitrary and just as independent of general slope as the other.

However, in passing from one of these extremes to the other, the river would pass through a stage where something like a general slope relation might be said to control the flow; and here it would probably have a larger discharge on the rising river and a less discharge on the falling river, each in some proportion to the rapidity of the rise or fall. And this is the only point in such a river where velocity-slope relations, even if they could be found, would have any application whatever to its phenomena.

It may be well, finally, to note distinctly that the physics of such a river may be thoroughly known and fully understood, but this would hardly be called hydraulics; for, in the main, it begins and ends in a system of observed facts, and conventional hydraulics throws not a ray of light on the actual proportion in which its elements should stand; they have all simply to be measured.

So much for the river which is not alluvial; what follows will deal strictly with the alluvial river, where the flow handles readily the material of its bed. In such a river, then, that limiting relation referred to, of its slope with its form, is an actual one. In simple cases, this is easily made a subject of experiment. If material from the bed of any such river is put on an inclined plane, a straight channel made through it, and if a given flow which will move the material is turned into it, it will reshape its section until it takes one and only one form for a given inclination.

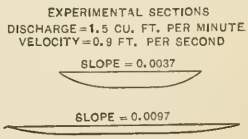

FIG. 2.

Two sections from such a series of experiments made by the writer are given in Fig. 2 to show this effect of slope on a straight channel. They serve very well to illustrate the wide difference in the types of different rivers and its cause from, say, the Platte River at one extreme, where, with a fall of some 6 ft. or more to the mile, it simply runs over the surface of the country, to

the Lower Mississippi at New Orleans at the other, where, with little or no slope, it maintains channel depths 100 to 200 ft. below the Gulf level.

But alluvial rivers are not straight, and neither would these experimental sections remain so. Such an equilibrium is simply a balance of forces, and is not necessarily the only one or the most stable one. Indeed, as the higher slopes are experimented with, the equilibrium of the straight reach is seen very clearly to be less and less stable. It still marks a well-defined point in the experiment at which the flow can be cut off and the section measured, but at the extremes there is really no time when the movement of material in the straight reach can be said to have absolutely stopped everywhere, and it goes on to the formation of bends and crossings, each with more or less accident in its location, with hardly a pause at this, its first and simplest but less stable form of equilibrium.

With a given discharge and slope, the equilibrium of the straight reach has the advantage of being everywhere an identity of form which may be reproduced readily and recognized easily. But though its final equilibrium is not an identity of form anywhere, and its accidental distribution of bends and crossings cannot be reproduced, as a whole, it is no less certainly an identity of effects for the same causes.

But, even with all the freedom in the variations of bend development which the final equilibrium offers, this is but the element of the actions which are going on in the alluvial river. For a given discharge, the bed is everywhere adapting itself to the given slope, while readjusting this slope over pools and bars to its most stable equilibrium, but the river in its season has all discharges from its high-water to its low-water extremes, and different extremes in different seasons; and as these all work each other over, and the results of all are overlaid in the existing conditions at any time, they very naturally produce the endless change of section and the infinite variety of form which are found in the river, the special studies of which are perhaps as fruitless as the study of the differences which may be found in the leaves of the same tree.

To measure this up with anything like thoroughness in a large river, in the first place, requires a great deal of field work, and then begins the difficulty of knowing what to do with it. The general type of the river and the geographical outlines of its bend development

are evident, while the distribution of slope resulting therefrom may be followed roughly through the different forms of the river in its pools and crossings. But, to go further and try to express it in systems of averages, or to follow the general method of compiling it all in the elements of cross-sections, can hardly be said to have been found to be satisfactory, either theoretically or practically. The phenomenon is some form of sequence which no known elements fully express, while to average things which are necessarily different is to obscure their meaning more or less.

However, it is important to note here certain facts in regard to cross-sections. On the Lower Mississippi, where they have been studied most thoroughly, it is plain that the cross-section at any point may not represent the bed of the river there within the limits of some 8 or 10 ft. That is, the mean elevation of the bottom on this line may have such a variation from time to time, while neither the general regimen of the reach nor the surface level of a given stage there have changed perceptibly. Perhaps a special term is needed here to indicate this bed of the river, which, for a given flow, certainly determines the surface level, and it will be called in this sense the "bed in train."

So long as the surface level of a given stage does not change in any reach of river, the bed in train under it certainly remains the same. But when the mean depths of the cross-sections are plotted as elevations of mean bottom on the same profile as this surface, they are generally found to be very irregular; they have sharp ups and downs of some 10 ft. or more, while the corresponding surface changes are measured in tenths or even hundredths of a foot. And, finally, with no change whatever in the regimen of the reach, when one or more of the same sections are re-sounded from time to time, and their mean bottoms show just a like variation of level, there is no room left for the least doubt that the data of sections has simply an accidental divergence from the bed in train of fully this amount.

Indeed, the more thoroughly the bottom of the river is known, the more this accidental variation, which really means nothing, is recognized. At high water sand waves 10 or 15 ft. high may be formed here, while there eddies may be cutting great holes in the bottom; and, with no change in the general conditions of a reach, the bottom at a fixed point in the Lower Mississippi may be found at elevations

differing by as much as 20 to 30 ft., and the mean bottom of a line across it by at least 8 or 10 ft. In smaller rivers these limits are, of course, correspondingly less, but, with their higher slopes, such changes may be even more sudden and erratic.

It is very evident, then, that the bottom anywhere, which at that point represents a real value of the sequence of phenomena in a river,

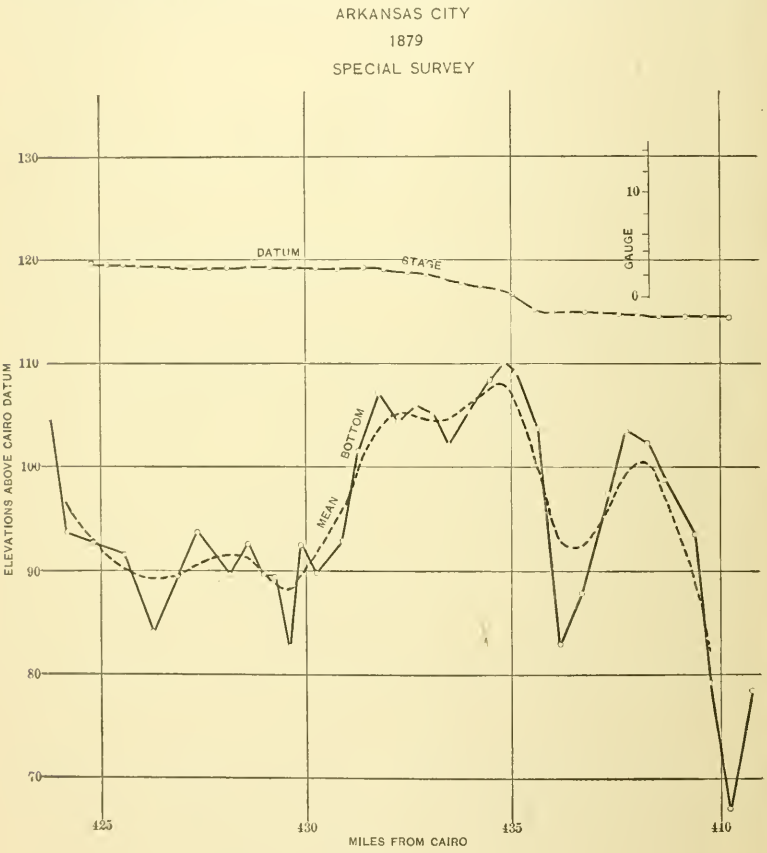

FIG. 3.

is something altogether larger than the bottom on a line there, say, half a mile across it, or a like line in any other direction. The local cross-section should doubtless have the most weight, but the bed in train is a part of the condition for some distance, both above and below it.

Some effort has been made to bring the cross-section data together in this form. Taking their elevations of bottom in continuous means of threes, dropping out an upper one and taking in the next lower one at each step, and again repeating the same process on these first means, generally succeeds at last in reducing it to a fairly regular sequence; and it certainly has the merit of bringing the results of a great mass of data together into a very simple form in which they can be seen as a whole and compared with other cases.

The results of a hydrographic survey on the Lower Mississippi in the vicinity of Arkansas City, shown in Fig. 3, will serve to illustrate this. The dashed line of mean bottom is the special sequence deduced by this process of lapped means, while the line joining the plotted points shows the local irregularities in the cross-sections. Each of these, however, in this case; is in general the average of two or more sections often with fifty or more located soundings to each; and, altogether, they have less of this accidental variation of bottom than any of the like data which the writer has yet worked over.

Either of these lines of mean bottom shows the pool and bar formation of this reach very well. It should be noted, however, that probably neither of them shows to its full height the crest of the bar which really controls the level of the upper pool at low water, while in the shorter bars of smaller rivers their failure in this respect would very probably be even greater.

This, however, is about as far as the direct study of form, in an alluvial river, has been carried, and, as before stated, it does not seem that much is to be gotten in this process of trying to understand the river from the bottom up. Practically, every step which the writer has made in river hydraulics, has been made in studies following exactly the opposite process ; but before taking up these, in which the whole point of view will be simply reversed, there is a generalization which the hydrographic surveys give, and which perhaps here it may be best to state.

This generalization rests on the observed fact that the average annual caving in of the banks, over hundreds of miles in some of the rivers, is at a rate which would altogether fill them up in a lifetime; and it follows that, for the river to maintain its channel in such a case, as it does, there must be going on annually some equal and opposite process of clearing this out. This simply considers a lateral action

whatever the movement of material in the direction of flow may be, when a reach, say like that of the Lower Mississippi from White River to Vicksburg is taken, nearly a mile wide and some 200 miles long, the difference between the material which comes in at the upper end and that which goes out at the lower would certainly have no appreciable effect on such a prism; while the bank erosion there is not far from a fill of 1½ ft. annually over this whole bottom. The clearing out of this quantity annually is, therefore, necessarily, the work simply of a lateral resultant.

How and when it may act will be better understood later. It is enough here to know that its action is a fact. It is not deduced from experiments on slope equilibriums, nor found directly in the forms of cross-sections. But, certainly in that light, much which has been said about alluvial rivers filling themselves up with the matter which they carry in suspension, is, to say the least, trivial. The difficulty which the water-works engineer has in getting the suspended matter out of the river water when once it has gotten into it, does not encourage the idea that any great additional load will be thrown on the lateral resultant from this cause.

While a river in all its upper part may be cutting down its bed, in the slow process of geological periods, undoubtedly in its lower alluvial part it may be building it up by the same process, and, with its alluvial part relatively short, this action may be even appreciable in the course of centuries; but in systems where the alluvial reaches extend up for thousands of miles, it seems hardly worth considering, and, certainly, not to the exclusion of the equally palpable fact that the alluvial river is also keeping its bed down, or otherwise its channel would fill up by the fall of material into it, and this also in periods which, in comparison with the other action, are relatively immediate

Discharge and Gauge Data.

Having come now to the point at which is recognized the difficulty of getting anything satisfactory out of the cross-sections to express for a given stage the bed in train it is proposed, in what follows, simply to take a surface measure for it. Laws of flow are there, whether or not they are known, and in these laws the bed determines the surface. Either can be taken as a measure of the other, and the question of how this measure is to be interpreted can be considered later.

In any case, if the flow into and out of a given reach is the same and the surface level is fixed, there is just so much water there all the time, and the bed, as a whole, is equally fixed under it. In the alluvial river this may be only an instantaneous equilibrium, neither the bed nor the surface may remain fixed, and the surface especially may be changing on account of changes in the amount of water flowing. There are also reasons for thinking that it may change on account of changes in the conditions of flow attending the rise or fall of the river, or the entrance of floods from tributaries; but if none of these will account for a change found in the surface, it must be accounted for by a change in the bed of the river under it.

The first thing required, however, to identify this surface measure, is to determine the flow which corresponds to it. If the river is on a stand through a given reach, with discharge everywhere the same, its surface level may be said to measure everywhere the bed under it; but this alone gives nothing which can be brought into comparison with any other part of the river, or indeed with the same river at another time. But when the discharge is also known, and the reach comes back, say after a flood, to the same discharge with a different surface level, this gives an absolute measure of the changes which have taken place in the bed of the river at that stage, though as yet this measure may not be interpreted.

At any location on the river, where a gauge record is kept, the readings give the surface levels from day to day through all the changes of the seasons, but to use these for the purposes in hand, or as given stages in that river, it is also necessary to know the discharges that go with them. This, of course, is the primary purpose of discharge data and the first thing to be considered in its study.

The observed values of discharge are plotted as abscissas to the gauge readings at the time of the observation as ordinates, and give what is called the discharge-gauge relation there. They show, as a whole, the increasing discharge capacity of the river from low water up, which may be expressed graphically by a mean curve drawn through the plotted points, or mathematically by an empirical equation. This, called in general simply a discharge curve, is taken for the scale of stage at that location. Some of the discharges may be below this mean curve, some above it. Barring errors of observa-

tion, these are different levels of a given stage, and mark either a change there in the conditions of flow or bed changes.

Of course no scale of stage in any river has an absolutely determined zero; the lowest low-water is as far down as it may be followed. In the same way, whether it is given by a graphic curve or an empiric equation, it has a physical limit at the top of the banks, or the level of overflow; it may in cases be more or less continuous above this level, or it may be altogether different. But between these limits it is a given absolute measure of all the average or normal physical and hydraulic properties of the river at that location.

How this scale is to be marked is still a matter of convenience. At any point in the river it is simply a range of level between low and

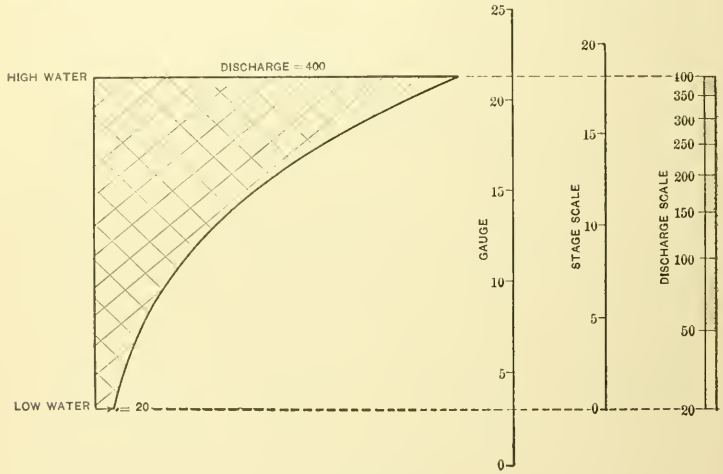

FIG. 4.

high water, with a given range of discharge corresponding to it. Drawn with a general characteristic of the Missouri River, but without any necessary reference to any river whatever, it is shown in Fig. 4, with some of the forms of marking which have been taken in different uses of it.

Where either the gauge or stage scale is used, the normal discharge curve corresponding to it is given at the beginning, and after that the stage is simply taken in feet and fractions with the given values of discharge understood. The discharge scale has the advantage of bringing together the two elements which define the given stage there, but it does not give their coincident values so accurately

everywhere. On it here the discharge is marked in thousands of cubic feet per second, a unit in which it will be generally expressed.

While each of these methods of marking the given stage has, as will be seen, its special field of uses, in the immediate study of the data, the gauge, of course, is the first one that is taken; but from the first, also, this must be clearly distinguished from simple gauge readings, for they have in them as well all the variations which may occur in the surface level of a given stage there through the flood season. It is true that, as this scale of stage differs materially from point to point in the same river, it may also change more or less from time to time at the same point, but what it is must be gotten before such changes can be followed, and it is important, in determining what it is, not to mix it up at first with all the sequence of bed changes and different conditions of flow which follow each other in infinite variety and endless succession from season to season. These have been called simply changes of plane, without regard to their causes, and in what follows they will be so designated.

It may be recognized here that the term "stage" is used, in a highly technical sense, as a standard of measure in rivers. Without a measure for force or for heat but little progress could be made in those sciences, and some definite measure is equally necessary in the hydraulics of alluvial rivers. In its general use, stage refers to that range in a river between high and low water, as high-stage, mid-stage and low-stage, or, in terms of gauge readings, 10, 20 and 30-ft. stages. But these are plainly very different things in a river say 100 ft. wide and another 1 000 ft. wide. This ordinary use expresses but one element of something which has at least another dimension. In terms of slope, area and mean hydraulic radius it has three, with none of them known and no known relations between them; but these are all summed in the coincident value of discharge, which, observed anywhere for one stage, is everywhere on the river for the same stage, the same value.

Expressed in this scale of stage, the different parts of the river may be brought into immediate comparison, and also the different types of different rivers; but as yet it has had no special reference to any particular river, and has hardly touched upon the data from which it is to be determined. The mean curve drawn through the discharge-gauge relation has been noted as giving it, but this alone forms only a

part of the data by which it is determined, and indeed a very small part in its extension over a whole river and the study of its general relations there.

Of course, this scale of stage gets its absolute value finally from discharge observations, but were not the relations in which all the different scales of stage stand to each other in different parts of the river first known, the study of discharge data might very imperfectly determine it; and these relations are almost wholly a question of the gauge data.

Between tributaries, whenever the river is on a stand, it is plain that its surface simply marks its points of equal stage on all the gauges; or, referring to Fig. 4, the low-water reading on the gauge is everywhere the zero of the stage scale with a corresponding discharge of 20. The top of the flood, in the same way, may mark the equal high-water stages, and intermediate stages are continually being given, between the varying rises and falls, in this direct manner.

In all this it will be found that the different scales of stage in the same river in general stand to each other in a simple ratio. That is, these readings, on any two gauges taken as ordinates and abscissas, give a relation between them which is a straight line. And further, in any rise or fall of the river, when a suitable time is allowed to let the difference of stage pass from the location of the upper gauge to that of the lower, the readings so taken mark the same line through the intermediate and recurrent stages and in all the periods of rising and falling river.

Fig. 5 will serve as a simple illustration of this. On it the Missouri series of discharge observations, taken in 1879 at St. Charles, are shown plotted to their St. Charles gauge readings, with the mean discharge curve given by them there for that season. With this also is given the relation between the gauge readings at St. Charles and those taken at Hermann, 69 miles up the river. In this the time interval of $\frac{3}{4}$ of a day is allowed for equal stages to reach St. Charles from Hermann, and interpolated St. Charles readings, this much later each day, are taken for ordinates to which the Hermann readings are plotted as abscissas.

All the readings on these two gauges, from early in April to the fall low-water in November, are shown in the plotted points of this gauge relation. And it is plain that for all values of discharge corresponding to the St. Charles stages, accurate corresponding Hermann values

may be taken immediately from this line. Or, the gauge relation is said to transfer the St. Charles discharge curve to Hermann, and this it does also with altogether about the same precision with which the curve is originally determined for St. Charles. Thus, in 1879 there are 170 discharge observations at St. Charles to give the mean curve there, while, at the same time, there are somewhat more than 200 entirely independent observations of the surface level at St. Charles and Hermann to fix this relation between them. Either determination has its possible

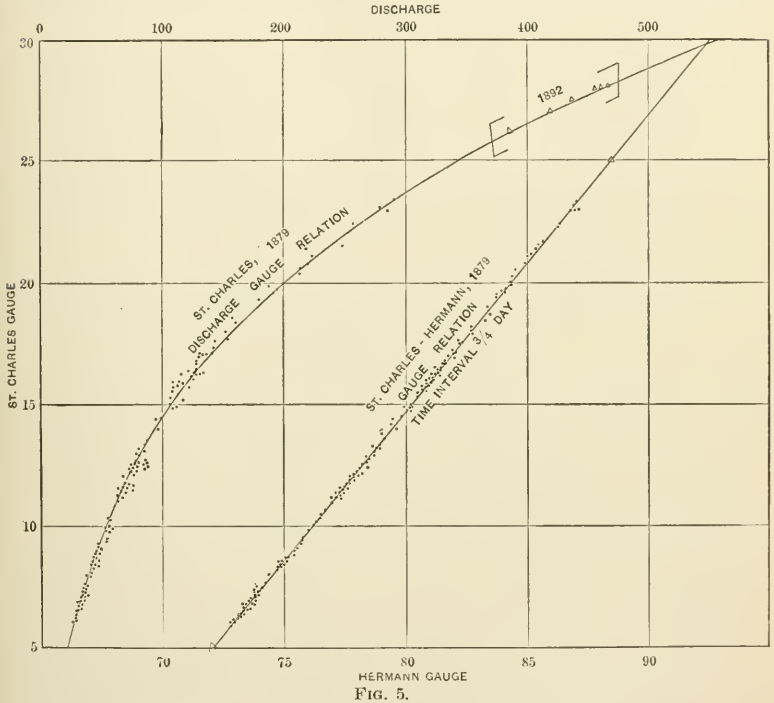

Fig. 5.

error, but the resulting scale of stage at Hermann may not have more but actually less of this error in it than the St. Charles determination.

As the St. Charles discharge curve is transferred to Hermann, so again it may be transferred from gauge to gauge along the river, meeting a positive test of both the original curve and of all the transfers at every other location where discharge observations have been also taken on the river in the same period. The only other observations made in this year on the Missouri were a short series at Sioux City, some 780

miles further up, and a transfer and check over this distance is a long, and necessarily a very careful, process. In passing such tributaries as the Platte or Kaw Rivers special studies of the gauge relations there for years are also necessary, and need either some discharge observations on the tributary, or discharges of the main river above and below it.

In fact, it is only in the same river that the relations between the scales of stage are straight lines, and "the same river" is, of course, to be understood as a physical term, and not a geographical name. The Missouri above and below the Platte need not have been the same river, but essentially it is, and this relation practically holds on the Missouri, at least from Sioux City to its mouth. Indeed, the smaller tributaries further down, in their low-water periods, have not more than 0.1 or 0.2 ft. of effect on the gauges below them, even at the extreme low-water stages of the main river, and this, of course, amounts to little or nothing at the higher stages.

However, a flood out of these tributaries is another matter. The Osage in extreme flood may add something like 150 to the discharge of the main river, and it is plain that in such periods, the line of relation, between the scales of stage above and below it, is no longer given by the gauge readings. The flood does not necessarily affect the scales of stage at either of the locations, unless perhaps the upper one is within the range of back-water action from the tributary, but the marked excess then of the lower gauge readings must show in a corresponding divergence from the line of the ordinary gauge relation, and, in fact, on the Missouri River, this is used as a measure of the floods from the tributaries.

Such cases are, of course, to be met with in plotting the gauge relations, and are excluded in these transfers of the discharge curves. Some of them come at inconvenient times, but long low-water periods are the ordinary conditions of these tributaries, and, using only such periods in general, the transfer may be carried across them without sensible error. In the same connection, also, the investigator will soon find that he has to deal with more or less gauge data which should never have been published. The effect of the tributary flood is continuous, and this fact shows in the gauge relations which follow it; but false readings are not, and by that they are marked at once on the gauges where they are made. The line selected for Fig. 5 was free

from all such complications. It is not at all an exceptional gauge relation, but it must not be assumed that all gauge relations are by any means so simple.

So far, the scales of stage have simply been taken as marked on the gauges, and, whether an original discharge curve, or a discharge curve transferred, they serve only as a measure of a more or less limited reach of the river at the given location. Before considering a more general condition, it is convenient to pass to the second form of marking shown in Fig. 4, and called the stage scale. For this, 5 ft. on the St. Charles gauge is arbitrarily taken as a low-water standard, and 25 ft. as a high-water standard, corresponding in this year to the discharge values of 19.6 and 347, respectively. In Fig. 5 the corresponding levels on the Hermann gauge are shown as determined on the line of that gauge relation. And taking these corresponding points from St. Charles to Kansas City on all the gauge relations between them, in all the combinations in which they may be plotted, gives for this year the equivalent gauge readings called low and high water, respectively, in Table No. 1, at the locations of each of the given gauges.

TABLE No. 1.

Location of gauges.	Mid-Bank Distances.	Year, 1879.		Equal stages (h) (high water).
	Miles above St. Charles.	Equivalent gauge readings.		
		Low water.	High water.	
St. Charles	0.0	5.0	25.0	20.0
Cottleville Landing	16.5	25.2	40.1	14.9
Washington	40.5	46.4	62.3	15.9
Hermann	69.0	72.0	88.5	16.5
Fisher's Landing	87.5	88.9	106.0	17.1
Jefferson City	112.2	110.7	127.5	16.8
Providence	136.5	134.2	150.0	15.8
Boonville	160.4	156.0	172.0	16.0
Glasgow	187.0	178.5	196.5	18.0
Miami	222.5	212.7	230.7	18.0
Waverly	249.0	233.7	251.0	17.3
Lexington	270.0	253.7	272.8	19.1
Camden	283.5	267.0	283.0	16.0
Missouri City	307.0	285.0	302.7	17.7
Kansas City	332.7	307.0	325.4	18.4

The differences between these arbitrary high and low waters, called equal stages, mark, of course, the scales of stage at each of these locations, mid-stage being simply the half of these values, and so on, for all intermediate stages. But the fact that these are 20.0 ft.

at St. Charles, and only 14.9 ft. at Cottleville Landing, 16.5 miles up the river, or 19.1 ft. at Lexington, with 16 ft. at Camden, 13.5 miles above it, shows distinctly the marked longitudinal variation which may be found in the river; and gives, at the same time, its definite measure. The whole length of the river is certainly a sequence of such variations, in its extremes at least as large as the differences found in these cases.

This sequence, in its actual longitudinal form at any time, has not yet been determined. But, while in certain bends it may stand unchanged for a long time, in general, it is not fixed. What was 20 ft. at St. Charles, in 1879, was but 18 ft. some five years later; in which case the river there had left its old channel around the St. Charles Island, and shortened its course by several miles. In the same way, the 16 ft. at Camden, in time, may take the place of the 19.1 ft. at Lexington. And this sequence, in its time variation, may be followed with precision in the discharge and gauge data. It is one of the hasty conclusions of a superficial study of this matter, that there are no gauge relations because they are not fixed ones, while, on the contrary, it would certainly be very surprising to find such a longitudinal variation in the river actually tied to all its gauges.

However, in the Missouri in general, and it is inferred in other northern rivers, there is really a period in which there are no such relations. Of course, when the river is frozen over there is not much rise or fall to mark any relation, but it seems that an ice-bound surface may result in a different distribution of slope from that of a free surface, and, in the place of the river standing at the same level on al the gauges, it may, in such a case, change in a perfectly arbitrary way at any of them. The clearing out of the ice also probably cuts up its bed pretty badly, as the forming and breaking of ice gorges might be expected to do. But certainly during and immediately following this period no relations between gauge readings may be found. That it soon readjusts itself and all its flow, in all the variations along its course, and, in all its changes from high to low water, is again found held in this chain of simple stage ratios, is not at all inconsistent with what has preceded it, and on the whole marks no less but rather more the presence of a determined and controlled equilibrium.

And, finally, the larger changes noted in the equal stages, which are in general a matter of years, must also be supplemented by

smaller changes which may be only a matter of seasons. Not only may the river change its whole course and its whole form at the location, but the bar may be built further down and bring the gauge from the level of a lower into that of an upper pool. The first may have an effect of several feet, the second may be limited to fractions of a foot, but on the Missouri the second may also mark several distinct gauge relations in the season, or throw a part of the plotted period out of the line of the general relation. These, of course, mean different scales of stage there for each period, but in such details the scales of stage are certainly restricted to very limited reaches. This might be of value in special studies, but, as a general river measure, the average is all that is wanted; and where there are two or more such changes in the season, a mean is taken for the gauge relation.

This outline will show the general processes and results of a study of discharge and gauge data. It gives an absolute measure of the river at a number of different locations, which probably cover all its variations, and follows with precision all its changes at each of them. To close for the Missouri, it is only necessary to add the fact that the Missouri in different years is a different river. Or, to state this technically, the level of the zero of its general stage scale is found to change materially between a sequence of high-flood and low-flood seasons.

Taking, for instance, the fall low-water of 1883, following a succession of phenomenally high-flood years, the low-water level corresponding to the discharge of 19.6, in the place of being the gauge readings from 5.0 at St. Charles to 307.0 at Kansas City, given in Table No. 1, would be found to average about 1 ft. lower. Whether or not the high-water stage is affected in this is still unsettled; from such data as there are on the subject, and from the nature of the change, it is inferred that it is not; in which case all the values in the column of equal stages would then average 1 ft. more. In the same way, the fall low-water of 1895, following a succession of phenomenally low-flood years, is about 1 ft. higher, giving values in the column of equal stages averaging about 1 ft. less.

The process of such changes will show later. They arise from the counter-actions, already noted, of erosion which would fill up the bed, and of that lateral resultant which would keep it down. Every cutting bank and bar is caving into the river continually, in

this way throwing upon the bed material which is not carried away, for there is no evidence of an increasing movement of material down stream; and indeed, accumulating as it would, it could not be thus taken up in the flow without soon overloading it; therefore, it must simply be spread over the bottom in some way, and this, at all times,

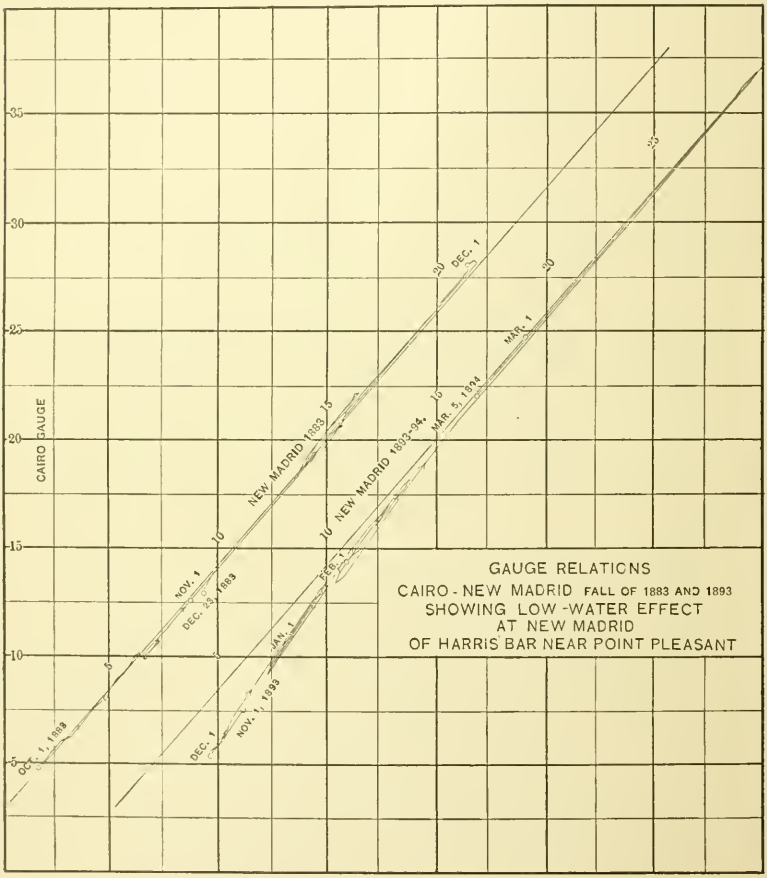

FIG. 6.

below the existing water line. It is really, then, only the flood which can move this material up on the slopes of points and bars; and the extreme flood is required to throw any of it altogether out at the bank level. Thus there is no doubt that the action of the series of extreme floods is this general clearing out of the low-water bed, and of the

series of low floods a corresponding filling up, notwithstanding the fact that this has not yet been observed in anything but these stage measures.

With all that has been given in this study of the Missouri, the subject may be followed now into the data of the Lower Mississippi; and it will be enough to note briefly the special features which the given processes show for this river, and at the same time any new problems which its marked physical difference and greatly enlarged scale may

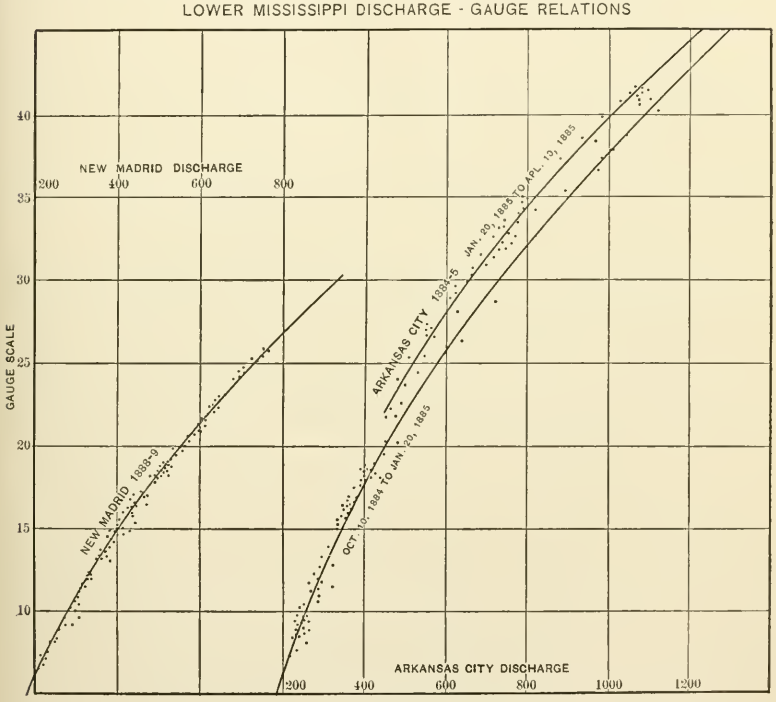
LOWER MISSISSIPPI DISCHARGE - GAUGE RELATIONS
GAUGE SCALE
NEW MADRID DISCHARGE
ARKANSAS CITY DISCHARGE
NEW MADRID 1888-9
ARKANSAS CITY 1884-5
OCT. 1ST, 1884 TO JAN. 20, 1885
JAN. 20, 1885 TO APL. 13, 1885

Fig. 7.

bring into view. And, certainly, between a river of the size of the Missouri, and one whose floods have some three times the extreme discharge and twice the rise, this matter should be fairly covered for alluvial rivers.

Figs. 6 and 7 serve as illustrations. The New Madrid discharge-gauge relation in Fig. 7 does not differ essentially from that of St. Charles on the Missouri, and it may be seen by the Cairo-New Madrid gauge relations in Fig. 6, that, in general, it may be transferred in just

the same way in which St. Charles was transferred to Hermann. But it will be noted here also that, while the relation at the higher stages has not changed perceptibly in 10 years, a decided but limited range of bar effect has come into the surface levels of the New Madrid low-waters. And it is this pronounced stability of these relations, some of which now go back nearly 50 years, subject now and then to these low-water variations, disappearing between the 10-ft. and the 20-ft. stages, which may in general be said to be the special features of the Lower Mississippi gauge relations.

This low-water form, however, is not a variable but a fixed condition in the 300 miles between Red River and the Gulf. Even as high up as Red River, the Gulf level is some 10 to 20 ft. above the mean bottom of the Mississippi, and through all this reach it acts upon the low-water surface in much the same way as did the crest of the bar in 1893 at New Madrid. For any such reach, of course, the variation of the scale of stage through it is a special and a single problem in each case.

But the marked exception of the Lower Mississippi gauge relations to the general features noted above, is found since the times, and at the locations, in which its great floods have been held within levees. It began to be seen, more than 10 years ago, along the swamp fronts, where the large natural outflow was being materially checked by the building of levees, that a general lowering of the low-water level was an accompanying phenomenon; and it was soon inferred that the levees there, producing abnormally large floods, like the series from 1881 to 1883 on the Missouri, were simply clearing out the low-water bed in the same process. Indeed, it was here that the action of that lateral resultant of the movement of material in a river was first suggested, even before the magnitude of erosion, determined between hydrographic surveys, had made it so plainly a necessity, and before the reverse process had been tested in the filling up of the Missouri River bed after the succession of low floods in 1895. Simply in these levee effects was it seen that, whatever might be the longitudinal movements in the building up of the bars at high water, it was, after all, the magnitude of the flood volume which kept down the level of the low-water plane; and the somewhat homely illustration suggested itself that, as a whole, the flood went through the river somewhat like a plow.

All these features of the Mississippi, however, raise no essentially new questions; but, the Arkansas City discharges, in Fig. 7, do. The

marked difference in the level of the discharges between the first and second periods is a change of plane of a character which must be considered here. If it was simply a permanent local change at that point, it would naturally be assigned to some radical change in the regimen of the reach, similar to that noted at St. Charles, when the range of stage there changed from 20 ft. to 18 ft. But, far from being such a permanent change, it is not even a local one.

As may be seen in the example given, the gauge relations on the Mississippi are not only especially stable, but they are also, in general, especially well-defined single lines; and, in that case, every variation which shows in the surface levels of the same discharge at one of the gauges, must show in the given ratio and the given period at the other. Changes of plane are transferred as well as discharge curves, and, as far as the gauge relations below Arkansas City, each shows a single line for both of these periods, this change of plane is necessarily an identity all the way down.

It is, of course, only actually identified at the location of the lower gauges; but where it is found repeating itself in this way, in both its ratio and period from gauge to gauge down, there can be little doubt that it is simply continuous between them. And in this process the change of plane shown at Arkansas City, is found to be a continuous one as far down as it can be tested accurately, and at least, in this case, for over 300 miles.

This change of plane coincides roughly with a flood out of the Arkansas River, it does not show on gauges above, and it is practically continuous through the rest of the flood and as far down as it can be tested through the rest of the river. While such a change of plane certainly holds a difference in the general slope from above, this difference does not account for it, for this difference is the thing itself. In the same way the difference of slope between the rising and falling river does not help matters, for this difference is as great at Helena as it is at Arkansas City. And, in fact, this change of plane is found here, as repeatedly elsewhere, to have much the same magnitude between the two periods whether the river is rising, falling or stationary in either of them. Indeed, as was pointed out at first, not much dependence is to be put in the direct slope-effect in the flow of rivers; it appears and disappears continually, as a general slope controls or does not control the flow.

Such a continuous change of plane, in the case in hand, has been considered, probably in part at least, a change in the conditions of flow accompanying the flood out of the Arkansas. And, for the entrance of floods from tributaries, there is this to be said, they certainly raise the surface level of the river above them without any corresponding increase of discharge. In cases in the Lower Mississippi this may be followed up with precision on the gauge relations; thus, floods out of the Arkansas River have been seen distinctly to raise the surface level of the Mississippi over 40 miles above by more than 1 ft. The flow of the main river, then approaching the tributary with its momentum somewhat reduced at a continually increasing rate, might, perhaps, be expected to carry a higher plane below for some time and some distance. And, indeed, in such a case as the run-out of the Lower Mississippi from Red River to the Gulf, where, even in the highest flood, only a small fraction of the water in the river is above the Gulf level, and the great mass of it has not a foot-pound of potential energy, such effects of floods from Red River certainly seem to have startling proportions.

In the years of great floods, also, changes of plane of this character, are marked from Cairo down, and, as changes in the conditions of flow, would be assigned to the large return flows and back-water drainage following the extended overflow. And, as they do not show here to any great extent in the low-flood years like that of the New Madrid discharges given, some such assumption is not altogether unwarranted. But, at the same time, it should be recognized clearly that marked bed changes also occur between these periods. The raise to the top of the flood has the effect of the erosion through the greater part of a season at its surface, with all the bar-building forces of the river under it, while this is practically reversed in the fall. And, until this movement of bed may be determined, it is needless to discuss further the special causes of changes of plane of this character.

Of course, with such changes of plane in the Mississippi, there is little doubt that changes of plane of the same character occur in the Missouri, but, if they are not large enough to be seen there, they need not be considered. The same change in the conditions of flow which would change the surface level by 1 ft. in a river from 50 to 100 ft. deep, would only change it by 0.1 ft. in a river from 5 to 10 ft. deep; and in the different type, the reduced scale, and much more active bed

changes of the Missouri, changes of plane of this character are simply unimportant phenomena. In the same way, again, if such studies were carried into a river like the Platte, all the importance of the Missouri River gauge relation might disappear in the small range and continuous bed changes of that river. Hydraulics is limited to relations which show; and perhaps it is altogether possible to study with a microscope, the flow in the simplest form of flume, in such detail as never to understand it.

Finally, all these detailed studies of the discharge and gauge data in a river may be put together in the discharge scales, and show at a glance all the floods of a river for years, with the combinations which make up such a flood as that of the Lower Mississippi.

Figs. 8 and 9 illustrate the use of the discharge scales. The regular gauge readings are plotted from day to day as ordinary hydrographs, and the discharge scales are drawn through the flood season from low-water up. Arranging these, then, in order down the river, all the contributions from the tributaries show in the differences between the flood volumes from gauge to gauge, and may be checked readily at any point with summations of the tributary increments taken from the gauge relations, while the combined floods of two rivers, such as the Missouri and Upper Mississippi, are seen summed up in the flood volumes of the Middle Mississippi at St. Louis.

In these diagrams simply the average discharge scales have been used, and no account has been taken of changes of plane through the period, but, of course, in checking up the final discharge curves in the different rivers it is necessary to take account of these changes. And there, indeed, the whole study of these data in the rivers meets one of its severest tests. If the sum of the St. Charles discharge and the Grafton discharge do not make the St. Louis discharge there is something wrong. Individual discharge observations may have their errors, but correct scale of stage can only have changes of plane and wrong gauge readings.

All this, however, may be followed, well enough for an illustration, in the average scales in Fig. 8. Thus, the top of the 1881 flood is a little less than 500 at St. Charles with a little less than 350 at Grafton, while St. Louis shows about 800. But the plane at Grafton is raised by an up-stream effect from the extreme flood in the Missouri, Grafton discharge observations at that time showing this distinctly, and, allow-

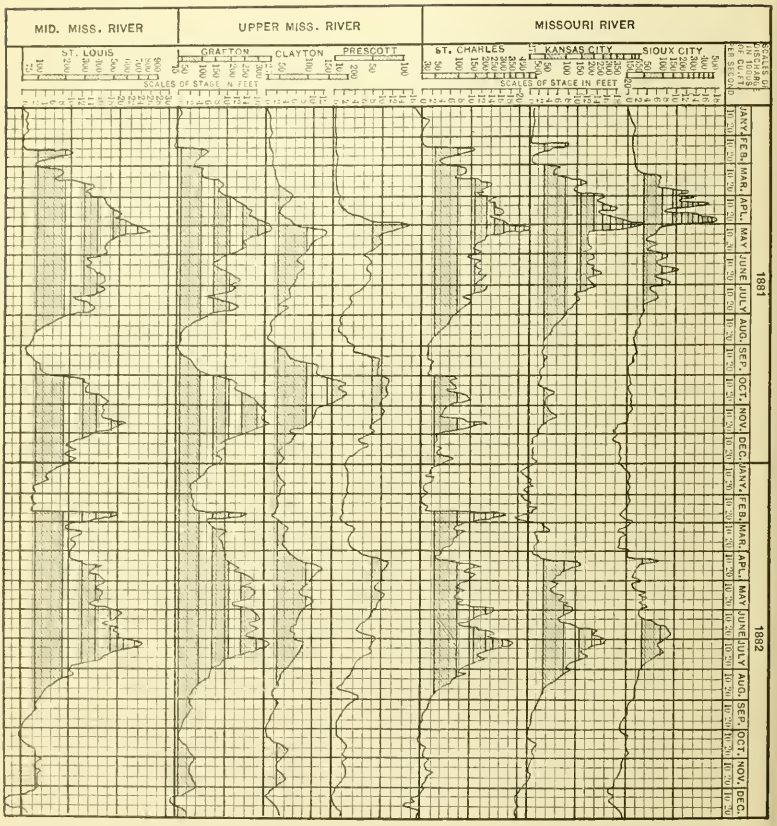
MID. MISS. RIVER
UPPER MISS. RIVER
MISSOURI RIVER
ST. LOUIS
GRAFTON
CLAYTON
PRESCOTT
ST. CHARLES
KANSAS CITY
SIOUX CITY
SCALES OF STAGE IN FEET
1881
1882
JANY.
FEB.
MAR.
APL.
MAY
JUNE
JULY
AUG.
SEP.
OCT.
NOV.
DEC.

Fig. 8.

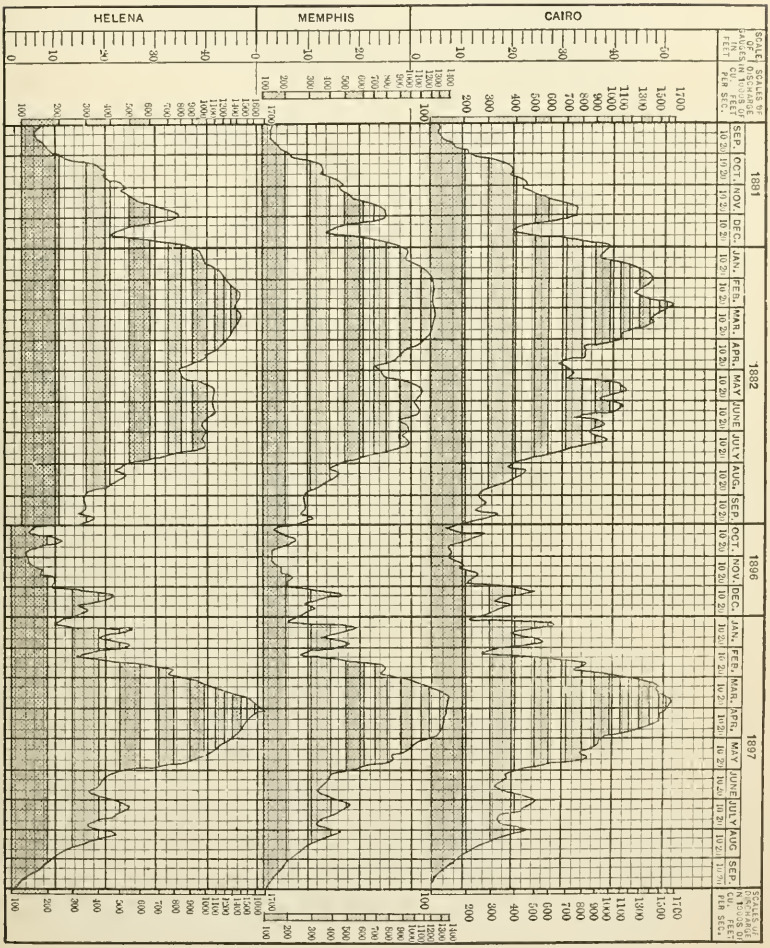
HELENA
MEMPHIS
CAIRO
1891
1892
1896
1897
SEP.
OCT.
NOV.
DEC.
JAN.
FEB.
MAR.
APR.
MAY
JUNE
JULY
AUG.
SEP.
SCALE OF DISCHARGE IN THOUSANDS OF CUBIC FEET PER SEC.

Fig. 9.

ing for this, it checks very well with St. Louis and the discharge from the Missouri. The next low water is 50 at Grafton, and a little below 50 at St. Charles, differing about the same amount from the 100 at St. Louis. And again, the top of the 1882 flood is 350 + at St. Charles, and 350 + at Grafton, with 700 + at St. Louis. At the same time, the irregularity of the ice period in these rivers is marked, and particularly in the forming and breaking of ice gorges from Sioux City down, in the spring of 1881.

Fig. 9 gives, in the same way, from Cairo to Helena, two of the greatest floods in the lower Mississippi; and Fig. 10 is an outline map showing the river's system of swamp basins. In general, all along these basins the ground slopes back from the river into an area of low swamps threaded with lakes and bayous, and draining into such rivers as the St. Francis and Yazoo.

As the Mississippi rises above its bank-full stage, the flood water is drawn off into these upper swamps, to be returned to the river at their lower ends after an interval. Fig. 9 shows this action through the first or St. Francis basin. The total volume of the flood, not only given by the Mississippi stages, but also traced down from its sources through the combinations of the upper rivers, is shown in the discharge scale at Cairo, while the same flood is shown at Memphis at about its point of maximum outflow, and again at Helena, where it receives the return flow from the St. Francis basin. However, the total return flow is not shown at Helena, since the outflow into the upper Yazoo at the same time is necessarily lost to its stage and discharge.

The former condition of free overflow is shown in the year 1882, and the first effect of the levee system now in progress in that basin is shown in 1897. And though the system was far from complete at the time, and was badly broken in the flood period, still the increase in stage and discharge is most marked. While the flood extreme of 1 700 at Cairo in 1882 is reduced by outflow to but little more than 1 200 at Memphis, and comes back with inflow to about 1 400 at Helena, in 1897 it reached about 1 400 at Memphis, before the breaks in the levee relieved it, and came back again at Helena to the Cairo value of 1 700.

This will illustrate the method of handling the extreme-flood data, which, as before noted, lie beyond the range of regular discharge curves. While not pretending to represent the actual discharge at any spe-

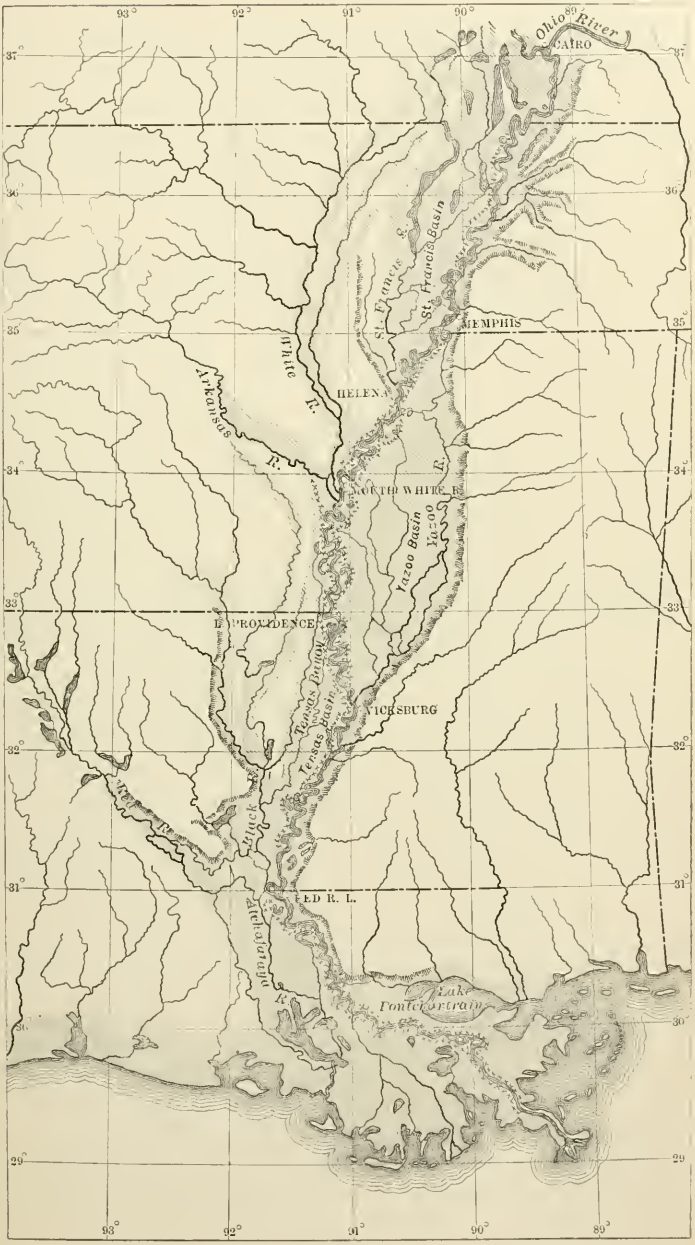
Ohio River
CAIRO
MEMPHIS
St. Francis R.
St. Francis Basin
White R.
HELENA
Arkansas R.
NORTH WHITE R.
Yazoo R.
Yazoo Basin
LITTLE PROVIDENCE
VICKSBURG
Tensas Bayou
Tensas Basin
Black R.
Ouachita R.
RED R. L.
Atchafalaya R.
Lake Pontchartrain
93°
92°
91°
90°
89°
37°
36°
35°
34°
33°
32°
31°
30°
29°

FIG. 10.

cial point in the flood, these scales, as an average, bring all the data together and compare their total volumes in a form in which some reasonable consistency may be recognized. However, further down the Mississippi, and especially since the construction of levees, the discharges are often so irregular, between the draw of a large crevasse on the one hand and the lake-like character of points in the general overflow on the other, that they can hardly be followed through the flood period, even in this system of rough averages.

Finally, Cairo and Memphis, in Fig. 9, show very well the fixed character of the general Mississippi regimen. The 15 years from 1882 to 1897, has brought no marked change there; while at Helena, in the same period, a case of the levee effect before noted is given. Here, in 1897, a glance shows that the discharge, as brought down to Cairo, from the rivers above, and as found repeated at Memphis, cannot be continuously some 50 less at Helena, as the 1882 discharge scale would give it. The precise lowering there is a matter of the gauge relation; but nowhere is the general fact more plainly seen than on the discharge scales.

In following the discharges down from Cairo to Helena through this period within banks, their readings on the discharge scales are essentially the same as they should be; for, in general, the tributary increments here would not show on these scales. Thus, the crest at the end of November, 1881, is a little less than 800 from Cairo to Helena, followed by a fall to just 400, and a rise to about 1 000, where the limits of overflow begin to show. But it must be noted also, that this does not mean that the actual discharges for the same scale readings in different periods are necessarily the same. The scale readings are normal values, and, so long as the actual discharges differ from the normal by the same amount all the way down, their normal readings will be the same.

This general identity from Cairo to Helena, therefore, is not only an identity of the discharge, but also an identity of plane. That continuous plane, noted in the case of the Arkansas City discharges, shows all through the data of the Lower Mississippi. It is referred to here again in order to note finally that whatever its causes may be, it has, from point to point down, exactly the same flood sequence preceding it, and its accurate recurrence from gauge to gauge only marks it as a very general and a very definite phenomenon; it does not add essentially to the difficulty in its explanation.

Flood Movement.

So far, it has simply been noted, in connection with the gauge relations, that, when the river was rising or falling, a suitable time was to be allowed for the given stage to pass from the location of the upper gauge to that of the lower. The precise determination of this time, from the gauge readings themselves, forms the subject of this study of flood movement.

In the rough, this time interval is seen very well in the hydrographs of any river simply by noting the difference in the times at which the flood crests show in order down on their respective gauges, and it is not only marked at the crest, but, somewhat less distinctly, at the foot of each rise. Thus, in Fig. 8, in the period May-June, 1881, on the Missouri, the general interval of about seven days from Sioux City to St. Charles is fairly indicated in the time between these points marked by the various crests and hollows; though it may be more or less blurred in the case of any one of them by the changes of plane and tributary increments which show in this period. In the same way the special retardation at the top of the April flood of that year will be noted. And, passing to the Upper Mississippi, its much slower rate of flood movement is shown most distinctly. Thus, comparing the movement from Prescott to Clayton with that from Sioux City to St. Charles, in the Mississippi, the flood takes about the same time to pass over about one-fourth the distance.

In the Lower Mississippi, Fig. 9 shows an interval of some three or four days from Cairo to Helena, and if any one should undertake to follow up this interval on the discharge scales, through all the rises and falls of the river, by noting the difference in time between the same discharges at Cairo and Helena, he would be struck at once with the general accuracy and the constancy of this interval, not only for the crests and hollows, but also for all the floods within banks.

But days are far from being the measure of precision which is wanted, and to obtain this, very different methods are needed. It is evident, on the face of it, that the time required for a given stage to pass from an upper to a lower gauge is determined with the greater precision the greater the difference of stage which is passing. Thus, the periods of rapid rise or fall are those which must be used for precision, while the period is hardly defined at all in the slower changes

of stage at the flood crests; though, taking these crests as a whole, symmetrically, it may still be seen in them. Almost any interval at the crest will give the gauge relations, while the line between any two gauges is given without any interval at all through all the stationary periods of the river which extend to both of them; but where a rise or fall of some 2 ft. a day occurs, an error of a tenth of a day in an assumed interval begins to make distinct divergences from the line of the gauge relation.

Having the line of the gauge relation given, and plotting such rises and falls with assumed trial intervals, the divergence of each plotted point from this line, of course, marks the error in the time assumption at that stage; but as, in the great mass of the data, the gauge relation and the time interval have to be determined together, it is convenient to have in mind, clearly, from the first, certain standard types which different trial intervals must give to the form of such a plotted relation.

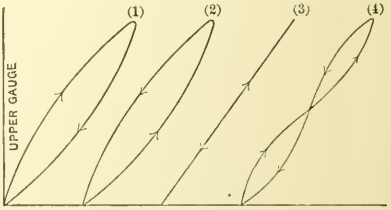

FIG. 11.

Types of trial time intervals are shown in Fig. 11. The first is an interval altogether too short. If, in plotting a trial relation, the assumed interval was less than the true time, then, on the rise, the lower gauge values taken would not have had time to rise to their equal stages, and would, therefore, be too small in proportion to the rapidity of the rise. In the same way, on the falling stage, the lower gauge values would not have had time to fall enough, and would, therefore, be too large in proportion to the rapidity of the fall. The trial relation would, therefore, plot as in (1) where the rising and falling stages are indicated by the directions of the arrows. In the same way, if the trial interval for the flood is taken too long, the lower gauge values would be too large for the rising and too small for the falling stages, giving the trial relation shown in (2). Between (1) the interval altogether too short, and (2) the interval altogether too long, the relation of the flood can always be reduced to one or the other of the forms (3) and (4). The form (3) shows very plainly that this interval for the whole flood is a constant, or that the flood movement does not vary with the stage; while all conditions of a variable

flood movement are included in the form (4). And here, in the special case indicated by the arrows, it represents a flood movement which increases about uniformly with the stage; that is, the mean interval is about as much too short for the lower part of the flood as it is too long for the upper part.

By more or less trial plotting of selected floods, the character of their movement, between any two gauges, may be recognized, and, in case the interval is a constant, it may generally be determined in this way to the nearest tenth of a day. It is also plain, however, that any flood out of a tributary entering between the gauges in this period would destroy altogether the simple form of the relation shown in (3). Indeed, unless such periods were recognized and excluded from the study, they would lead to very serious errors in both the rate and the character of the flood movement.

It is also necessary to allow for those changes of plane which may occur from time to time at the location of one or the other of the gauges. To determine the time required by a given stage to pass through the reach it must be taken on the level which it has in passing. So long as the levels of the given stage remain the same at the two ends, or change together, as they do through those continuous changes of plane noted on the Lower Mississippi, the relation will give the form (3) when the correct time interval is taken, but when the level of the given stage changes at one of the gauges without a corresponding change at the other, the form of the true gauge relation should shift in the same way; and an interval which might make it approach more closely to the form (3) would not then be a more correct interval, but an erroneous one.

These, of course, are the same variations which have been met in the transfer of discharge curves, and which, with a little care, do not affect sensibly the accuracy of that process. But here, in the determination of an element which is marked alone in the form of special periods of rise and fall, they are much more serious difficulties. Thus, taking gauges above and below a tributary, after getting by trial a correct interval between them, and then, with this interval, systematically plotting their relation through years, a number of the floods might be seen to correspond very closely with the form (3), and a number of others might be marked clearly as affected by floods from the tributary, or changes of plane, and rejected. Between these two

however, there would still be a number of periods in which it would be uncertain whether their form did not measure as truly a flood movement there as did the form (3), and in which, if they did, it would be plain that the flood movement, in that case, was not at the same rate as the other, nor indeed any longer a constant in its character.

This, of course, would have been all well enough if it had proved that the flood movement varied from time to time and for different forms of floods, but when the precision of the determination is also considered it is at once seen that in itself it simply does not prove anything. There is no use in taking interpolated values between the daily gauge readings any closer than to the nearest tenth of a day; if for no other reason, because it is doubtful whether the observers have actually read their gauges closer to the set time, if indeed they can be counted on for that. With an interval then of some two or three tenths of a day between the gauges above and below a tributary, this relation showing in any case the form (3) means only that the flood movement at that time is this given rate and is constant within the limits of some $\pm$ 20%, while in the other case it means that its rate is something more or less and variable within the same limits. Thus, the two cases simply are not precise enough for their contradictions to mean anything.

Again, while the short reaches lack precision in this time element, the long reaches lack precision in the equal stages. When the river is rising or falling 2 ft. a day, a change of 0.2 ft. in the stage is equal to a tenth of a day in the time interval, while in periods of less rapid rise and fall the time equivalent is correspondingly greater. In a long reach, therefore, covering a number of tributaries, the summations of their increments, small in themselves, may affect materially the apparent time of the passage of a given stage through the reach; and even where they may not do this, their combinations tend to curtail greatly the periods which are practically free from this difficulty, until finally, it is not once in years that a suitable period for the determination of its flood movement can be found; that is, a period without change of plane between the gauges, all the tributaries being at a fixed low-water, with a sharp rise and fall of the main river.

To meet all these difficulties in the precise determination of the flood movement, the system of plotting the data, called extension-gauge relations, was planned. Its principle consists of studying a

flood movement in the form of a number of gauge relations, between the same upper gauge in each case, and lower gauges following in order down; showing, in close succession, the movement of the same flood from the same start, through a series of successively lengthening reaches. The form of each gauge relation, then, in its order, includes all the flood movement of the relations preceding it, with such additional movement as the flood may have in that case between its lower gauge and the gauge next above it.

An instance from the Lower Mississippi extension-gauge relations is given in Fig. 12, covering a reach of 277 miles of that river, from Cairo down. In all this the time intervals have been determined first by trial. Cairo gauge readings are taken uniformly as the ordinates,

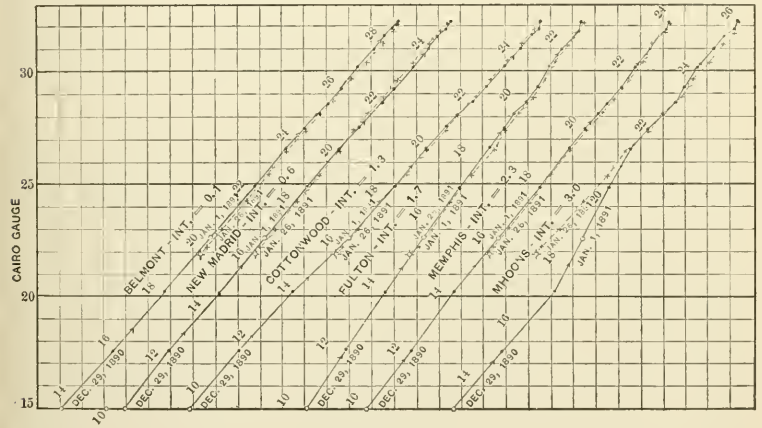

Fig. 12.

and interpolated readings, later by the intervals, respectively, as abscissas, and the gauge relations, Cairo-Belmont, Cairo-New Madrid and so on, to Cairo-Mhoons, are plotted in succession. Altogether, they leave no doubt whatever as to the movement of this flood. The Cairo form is simply reproduced in a given ratio on each of the gauges after the lapse of a fixed time.

Gauge readings, of course, contain a tenth or so of accidental error, and no single one of these relations reproduces exactly the Cairo form in the given ratio at the location of its lower gauge; but when the divergence is in one direction at one gauge, and in an opposite direction at the next, there is certainly no change of form shown in

the passage of the flood through the reach; and, indeed, in this case it is repeated even more exactly at Memphis, 230 miles below Cairo, than at Cottonwood 123 miles below, or Belmont only 21 miles below.

Thus, it is not necessary, in every case, to see this flood movement extending down to Memphis, or to Mhoons, in order to recognize it. If it is seen extending down to Cottonwood, and if, from there on, more or less increment from tributaries comes in to break it up, it is still seen, and the reason that it does not show further down is recognized. Thus, a part of the rise at Fulton may be marked with an excess of Fulton stages, and in that case the same part at Memphis and Mhoons would at least be marked with the same excess, though more probably it would actually show increments somewhat greater. Through all this, however, there would be little doubt that the same flood movement was still there, if outside conditions had only left the same flood in the river to show it.

The Mhoons' relation is given here as an instance which needs a special notice. On January 1st (January 4th at Mhoons), the gauge reader reported that his gauge was washed out, and that readings were taken on temporary gauges. This continued until May 23d when his gauge was reset, the inspection showing that his readings were 2.2 ft. too low. It is very evident from the relation that this error was made at the first setting of his temporary gauge, and if it is so taken the flood movement is shown to be very fairly continuous to Mhoons. But if, on the other hand, the error had been distributed over the whole period up to the inspection, as is often done before the gauge readings are published, it would have given an altogether incomprehensible surface movement of the river there for about five months. This is simply a caution which should be borne in mind continually in any use of the gauge data.

It is this process of recognizing, in the extension-gauge relations, what does not, as well as what does, show the flood movement, which gives all the certainty to its determination from the ordinary gauge data. The method may be said to be almost mechanical. The intervals of the various reaches are determined by trial. Stages between the readings of the lower gauges are then interpolated by these intervals, and are each, for the given day, the effect of the given reading on the gauge at the head of the reach, in so far as this has been

determined approximately by trial. These are then plotted in succession, showing the whole surface movement over hundreds of miles, everywhere as nearly in the one relation of cause and effect as a single time adjustment will give it ; and all, for the better part of a year, are brought finally under the eye at once, and in quick succession a number of years can be examined. After that the whole matter is simply one of intelligent seeing.

Such a flood movement as that of Fig. 12 is seen frequently and through long periods, extending over the shorter reaches, but with little precision in the time intervals; while, again, through the long reaches, with the much smaller percentage of error in the intervals, the flood movement can be followed only through shorter or less frequent periods. But as far as it is so recognized and followed, so far it is determined. Its actual rate is not given precisely between any of the gauges, but it is there, or it would not be found precisely in their summations of it.

In the case given, the movement may be said to be recognized down to Mhoons, and the interval there determined to the nearest tenth of a day, or to within ± 0.05 in a total interval of 3 days. This is less than 2%; and again, where the same movement is recognized down to Helena, or through a total interval of $3\frac{1}{2}$ days, the limit of error is then reduced to less than 1.5%; and noting repeatedly just such a flood movement in different floods and through all the different stages within banks, it comes to be held as a very general phenomenon of the river, observed with a good deal of precision, and certainly determined more closely than might have been expected simply with interpolations between daily gauge readings.

This, however, is all a case of the constant interval, or the flood movement which does not vary with stage. Type (4), Fig. 11, or the flood movement which varies with stage, is recognized and its form is followed in the extension-gauge relations in much the same way. That is, as the further a flood goes which does not change its form, the more certainly is it seen that its form is unchanging; so, in the case of a flood which changes its form, the further it goes the more distinctly this shows itself in its greater divergences from the standard form at the head of the reach. Thus if type (4), Fig. 11, showed a perfectly uniform increase of flood movement with stage for 100 miles, in 200 miles it should everywhere show just twice the divergence.

However, the case of a variable flood movement is never quite so simple, for it represents a condition which cannot continue indefinitely. Thus, the upper part of a flood cannot go on moving much faster than the lower part without coming to a point where it would topple over; and, before it reached this, any uniform or normal movement which it might have at a given stage would certainly be modified by the abnormal form which the flood was taking.

It is this fact which makes the determination of the flood movement in the Missouri, from the ordinary gauge data, so intricate. In the Missouri it has probably as definite a normal character as that of the Lower Mississippi, but this, in the Missouri, is a very decided increase with stage, and the normal movement, therefore, is modified at least by limiting forms of the floods, and is subject to variations with their changes. This, with the fact that interpolations between daily gauge readings are necessarily a less precise surface measure in its short and sharp floods, and that its tributaries through long reaches have a much larger proportionate effect on the main river, all taken together, deferred anything like the setting of values for the normal flood movement in the Missouri to a much more advanced stage of the study.

It is, however, in this contrast that it should, perhaps, be noted that a given flood movement in a river is not a law but a fact. Even with the general character of the constant interval on the Lower Mississippi, there are still a few specially sharp floods and some short periods in the transitions from stationary to flood conditions which seem to be exceptions. But that does not detract at all from the significance of the fact that its whole surface movements from low water up for almost its whole time are certainly held in this given fixed relation.

As simply an empiric constant, running through the infinitely complex and ever-changing conditions of flow, this recurrent flood movement seemed to be the one thing in that river which did not change; but why it should be of the same rate at high water as at low, while the mean velocity of the river for the same change about doubled its value, or why it should be over 400 miles a day from Baton Rouge to Carrollton when even the high-water velocities there were less than 100, were questions at the time well calculated to puzzle anyone; for probably everyone would first try to explain this flood movement by the direct movement of the water down the river.

Such considerations may have some value in finally knocking out of the investigator himself any remnant of a delusion that water moves in threads and filaments; but as these considerations lead to nothing in the matter in hand they are here omitted. This flood movement is simply taken as it is, a well-determined fact of the river, which, as a fact, naturally rests on some property of the river as a whole which is altogether as general and as strikingly fixed as the flood movement. Anyone is, of course, at liberty to try and find this property in observed velocities if the total independence between the two, in the cases just cited, does not satisfy him.

Yet, with all the river absolutely determined in its scales of stage, and such a property of the river as disclosed by this flood movement also given, it seems certain that somewhere there is to be found some equally absolute relation between them. But seeing this and getting the relation are quite different matters, and it was not until the writer had studied analytically the case of flood movements in a chain of lakes or reservoirs* that he began to be able to calculate some of the elements of a river from its given discharge curves and flood movements. The methods at first were crude and laborious in the extreme, and need not be considered here further than to note the final step in this chain of investigation, for from here on the question will be taken up as an immediate deduction of a fundamental physical equation of rivers, with simply examples of its application.

Mathematical Analysis and Computed Regimens.

It is plain, of course, that for a given difference in discharge between the upper and lower ends of any reach, the rate of rise or fall in the reach is determined solely by the area of its water surface at that time. This general relation is given by the equation.

$$(Q_1 - Q_2)\, d\,T = A\,(d\,h) \ldots\ldots\ldots\ldots\ldots\ldots \text{I}$$

where Q_1 = the upper and Q_2 = the lower discharge in cubic feet per second, T = the time in seconds, A = the surface area in square feet, and $(d\,h)$ = the mean rise in the reach on a scale of feet in the differential interval, $d\,T$.

So far, this simply expresses the fact that a cubic foot of water requires a cubic foot of space, and expresses the condition of the whole of a river as it does every other form of variable flow. For the

* *Transactions*, Am. Soc. C. E., Vol. xl. p. 355.

river, the special problem is to express the terms in this condition in elements which may be determined independently.

Now, in whatever condition of rise or fall any river may be, $\left(\frac{d\,Q}{d\,h}\right)$ is the rate of change of discharge with its change of stage, and $\left(\frac{d\,h}{d\,T}\right)$ is the rate of change of stage at any point of its flood oscillations. The change in the value of discharge in an assumed interval $\Delta\,T$ is then $\left(\frac{d\,Q}{d\,h}\right)\left(\frac{d\,h}{d\,T}\right)\Delta\,T$. Now, whatever this may be on either rise or fall, there is some point down stream at a distance $\Delta\,l$ where the value of discharge at the end of the interval $\Delta\,T$ is just what it was at the upper point at the beginning of the interval. The difference in discharge between the upper and lower ends of the reach, of length $\Delta\,l$, is then $\left(\frac{d\,Q}{d\,h}\right)\left(\frac{d\,h}{d\,T}\right)\Delta\,T$; and the surface area of this reach is $W\,\Delta\,l$, where W is its mean width in feet. Substituting these, respectively, for $Q_1 - Q_2$ and A in equation I gives

$$\left[\left(\frac{d\,Q}{d\,h}\right)\left(\frac{d\,h}{d\,T}\right)\Delta\,T\right]d\,T = W\,\Delta\,l\,(d\,h),$$

or

$$\left(\frac{d\,Q}{d\,h}\right)\left(\frac{d\,h}{d\,T}\right) = W\,\frac{\Delta\,l}{\Delta\,T}\,\frac{(d\,h)}{d\,T}.$$

In this form $\left(\frac{d\,h}{d\,T}\right)$ and $\frac{(d\,h)}{d\,T}$ are not necessarily identical, for $d\,h$ in the first is the rise in the interval $d\,T$ at the head of the reach and $(d\,h)$ is the mean rise over the whole reach during the same interval. As, however, the corresponding magnitudes of $\Delta\,l$ and $\Delta\,T$ are reduced, the difference between $\left(\frac{d\,h}{d\,T}\right)$ and $\frac{(d\,h)}{d\,T}$ may be made smaller than any assignable quantity and finally disappear from the equation as a differential when $\frac{\Delta\,l}{\Delta\,T}$ becomes $\left(\frac{d\,l}{d\,T}\right)$.

Hence,

$$\left(\frac{d\,Q}{d\,h}\right) = W\left(\frac{d\,l}{d\,T}\right).$$

Now, by its definition here, $\left(\frac{d\,l}{d\,T}\right)$ is the flood movement at that point of the river, or the rate at which equal stages or discharges move down; and, representing its value in feet per second by m, the equation becomes

$$\frac{d\,Q}{d\,h} = m\,W \qquad \text{II}$$

It may be noted, in the above process, that the parentheses ( ) have been used to indicate definite physical values determined independently. Thus $\left(\frac{d\,Q}{d\,h}\right)$ is given by the discharge curves. On the Lower Mississippi it may vary from 10 000 at low water to 60 000 at high water; or in the Missouri from 3 000 to 40 000; and it is commonly referred to anywhere as the change of discharge per foot of rise or fall there. Again, $\left(\frac{d\,h}{d\,T}\right)$ is the rate of rise or fall at any time. It is in common use as feet per day or inches per hour, but, being expressed here in second units, its value, for a rise of 2 ft. per day, is $\frac{2}{86400}$. And, finally, the $\left(\frac{d\,l}{d\,T}\right)$ is that rate of flood movement which was at

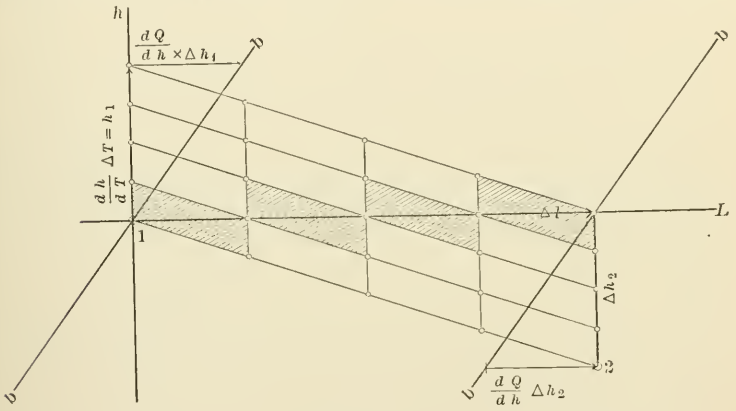

Fig. 13.

last determined with precision, at least for long reaches, in the preceding study of that subject.

With these in mind, the process of deduction is simple enough, and it is seen readily that Equation II is a most general equation of rivers, true for every element of any river from high to low water and from one end of it to the other. But, before going further, it may perhaps be as well to give, in addition, a somewhat more distinctly physical deduction of it.

Taking then, as in Fig. 13, an origin O_1 on the surface at the upper point at a given stage, and the axis of L a length of the river below at the same stage; its level representing the surface slope of the river when the discharges are equal at the two ends, or the river is on a stand

all the way down; then, with the ordinate h measuring the change of stage from this stand, the actual surface of the reach in any given rise is shown by the line $O_1\ O_2$. Taking now $b\ b$, an element of the discharge curve for the reach, its absolute value everywhere at the level $O_1\ L$ being Q_1, and the rate of its change of value $\frac{d\ Q}{d\ h}$, then, if $\Delta\ T$ is the interval which it takes to rise from O_2 to the level $O_1\ L$ and $\Delta\ h_2$ the corresponding change of stage there, at the beginning of this interval the upper discharge is Q_1 and the lower is $Q_1 - \frac{d\ Q}{d\ h}\ \Delta h_2$, the difference being $Q_1 - \left(Q_1 - \frac{d\ Q}{d\ h}\ \Delta\ h_2\right) = \frac{d\ Q}{d\ h}\ \Delta\ h_2$. In the same way, at the end of the interval $\Delta\ T$, the upper discharge is $Q_1 + \frac{d\ Q}{d\ h}\ \Delta\ h_1$ and the lower discharge is Q_1, the difference being

$$\left(Q_1 + \frac{d\ Q}{d\ h}\ \Delta\ h_1\right) - Q_1 = \frac{d\ Q}{d\ h}\ \Delta\ h_1.$$

The mean difference in the discharge between the upper and lower ends of the reach, through the interval $\Delta\ T$, is therefore $\frac{d\ Q}{d\ h}\left(\frac{\Delta\ h_1 + \Delta\ h_2}{2}\right)$, and there is thus brought into the reach during this interval a quantity $\frac{d\ Q}{d\ h}\left(\frac{\Delta\ h_1 + \Delta\ h_2}{2}\right)\Delta\ T$ more water than is carried out. This must fill the volume indicated by the vertical area $\Delta\ l\left(\frac{\Delta\ h_1 + \Delta\ h_2}{2}\right)$ multiplied by the width of the river W.

And equating these two

$$\frac{d\ Q}{d\ h}\left(\frac{\Delta\ h_1 + \Delta\ h_2}{2}\right)\Delta\ T = \Delta\ l\left(\frac{\Delta\ h_1 + \Delta\ h_2}{2}\right) W;$$

or, as before,

$$\frac{d\ Q}{d\ h} = W\frac{\Delta\ l}{\Delta\ T} = m\ W \ldots\ldots\ldots\ldots\ \text{II}$$

The application of this equation may perhaps be best noted by bringing it into comparison with the ordinary hydraulic formula

$$r = c\sqrt{r\ s} \ldots\ldots\ldots\ldots\ (1)$$

$$\frac{d\ Q}{d\ h} = m\ W \ldots\ldots\ldots\ldots\ (2)$$

Thus (1) and (2) have in their order essentially similar physical measures. For instance, r in the river is obtained by measuring the

discharges from high to low water and dividing them by the corresponding areas of their cross-sections; while $\frac{d\,Q}{d\,h}$ is obtained by plotting them to the corresponding gauge readings and differentiating the resulting curve. In the same way, s and m are taken from surface observations; one being a rate of changing level, the other a rate of advancing stage; while r and W are simply characteristic lengths, the first a sort of inverted radius, or area divided by wetted perimeter, the second a plain surface width.

Finally, also, in applying the formula $v = c\sqrt{r\,s}$ to a perfectly general form of flow, where everything may have any arbitrary variation in a given length, subject alone to this condition, even if it expressed everywhere such a variation and the c in it was an absolute constant, it would still be very evident that the element through which it must be taken is not a uniform length element. Thus, if it is to be summed into an equality and averaged, the given length is to be divided into a certain number of elements, each $v\,\Delta\,T$ long, and in each of which it expresses an equality, and the general relation of the whole distance is the average of these elements. That Equation II has exactly the same characteristic may be seen readily by taking it before its reduction, as $\frac{d\,Q}{d\,h}\,\Delta\,T = W\,\Delta\,l$. For $W\,\Delta\,l$ is simply the surface area in the variable elements of length, and $\frac{d\,Q}{d\,h}\,\Delta\,T$ corresponds to the $v\,\Delta\,T$ for each of the constant intervals of time into which the distance may be arbitrarily divided.

Here, however, the parallel ceases, the ordinary hydraulic formula refers in the main to conditions in which the discharge is fixed, such as the discharge of a pipe under a given head, or a conduit with a given form and slope; and it is certainly a most necessary relation there, notwithstanding the fact that its c is a very intricate variable, for Equation II has no application whatever to such a case. But, on the other hand, in rivers Nature offers a condition of an almost continually varying discharge, and, as there is no way by which an irregular volume can be measured more accurately than by taking the amount of water required to fill it, in such a field the fundamental relation between dimensions and the action on them of these varying in-flows and out-flows is not only applicable, but is the only one, in the range which may be covered by it, to be considered.

But, of course, this is not the place to discuss the application of either of these formulas to all the conditions of flow to which they may be applied. And, in fact, in the case of Equation II, there are probably a number of applications which the writer has never yet thought of. He has been interested mainly in applying it to alluvial rivers, and even there, in those reaches in the Lower Mississippi, where the gauge relations are no longer straight lines, it has been already intimated that its application in each case was a special and a single problem. In the whole of this lower river also, with its peculiarly constant flood movement from high to low water, marked with decided differences from reach to reach, there are some questions on which the writer is still working. The only case, therefore, which will be offered here is its application to the general regimen of the Missouri from Kansas City to St. Charles.

The 1879 scales of stage have been given for fifteen locations on this reach, in the former study of discharge and gauge data, and whether they should cover equal time intervals, or simple intervals of distance, is not here a practical question, since their longitudinal sequence has never been determined on this river, and a plain average of them is all that can be obtained, and also, very probably, all that is here necessary.

The average of the equal stages in the tabulation referred to, is 17.1, and the equation of the St. Charles discharge curve in terms of its stage scale is $Q = 0.15\ (h_s + 19)^4$ where Q is the discharge in cubic feet per second and h_s is the St. Charles stage taken from a zero at the low-water level of 5 ft. on that gauge. Calling h_m the average stage of the reach from Kansas City to St. Charles, it is plain that

$$h_s : h_m = 20.0 : 17.1 \text{ or } h_s = \frac{20.0}{17.1} h_m$$

and substituting

$$Q = 0.15 \left(\frac{20.0}{17.1}\ h_m + 19\right)^4$$

from which

$$\frac{d\ Q}{d\ h} = 1.117\ (h_m + 16.3)^3.$$

The corresponding value of flood movement on the Missouri has not yet been given; and indeed there were a number of questions in flood movement which were purposely left to be answered after Equation II had been determined.

Solved for this element, it is $m = \frac{1}{W} \times \frac{d\,Q}{d\,h}$, and with this it only takes a glance at the discharge scales in Fig. 8 to see why the flood movement is so much slower in the Upper Mississippi than in the Missouri. In the same way the marked retardation at the top of the great flood of 1881 on the Missouri is simply the transition from channel filling to valley filling. That the value of W changes sharply in such a transition is plain, while the especially irregular forms of the over-flow discharge curves have been noted. And, in fact, it seems probable that if these forms are ever to be determined satisfactorily it will not be done by measuring their discharges, but by measuring the widths of over-flow and the flood movements, and calculating them.

Again, the 400-miles-a-day flood movement from Baton Rouge to Carrollton is simple enough to anyone who knows the physics of that part of the river. It is well down in the run-out, where the range of high water is being reduced from 50 ft. at Red River to nothing at the Gulf; and the value of $\frac{d\,Q}{d\,h}$ is correspondingly increasing; while at the same time it is also an exceptionally narrow river. It is clear, then, from the equation, that values of m will be found here out of all proportion with its rates elsewhere.

And finally, the constant rate of flood movement from low to high water in the Mississippi will be recognized here as simply a special relation between its general discharge curves and its form. So long as $\frac{d\,Q}{d\,h}$ and W vary in the same proportion m must be a constant, and that they have such a variation vertically in the Mississippi is a fact of its form, and that they do not have such a variation there longitudinally is another; while in the Missouri they have no such special variation together in either direction. But still, all the Missouri scales of stage are linked together in a chain of gauge relations, and there is little doubt that this river has fully as definite a form of its own to correspond with the $\frac{d\,Q}{d\,h}$ given by them, and that its own especial flood movement is there, as certainly as it is in the Mississippi, even though it cannot be determined so accurately in its gauge data.

There is, however, a special variation of m which must be considered. It has been seen, of course, in the deduction of Equation II, that the elements of rise or fall, of whatever magnitude, cancelled out in the final equation, leaving a relation which was immediately

independent of them. This does not mean that the value of m is absolutely independent of the rate of rise or fall, but simply that it is changed by it only as the $\frac{dQ}{dh}$ may be changed by it, assuming, at the same time, that the element of form W at the given level remains the same.

This is a variation of the flood movement from its normal, of the same character as a change of plane in the discharge data, and probably this is the only surface measure in which all those variations of discharge at the given level can ever be followed. It must be looked to also to account for those exceptions to the general flood movement noted in the Lower Mississippi, and the much more frequent changes of the same character in the Missouri. But it should be distinctly recognized simply as a variation from normal flood movement, and it does not coincide with the normal discharge or the normal $\frac{dQ}{dh}$ determined for the river.

It was in this light that the specific determination of the normal flood movement in the Missouri was taken up. The extension-gauge relations from Kansas City to St. Charles had all been plotted for a period of 15 years, showing the character of the whole surface movement there; but it did not seem worth while then to go further in order to pick out the values of its flood movements in all their variations. But when it was understood very clearly that there was a normal movement there which rested on its general form, already so distinctly marked in its discharge scales, the case was different. The extension-gauge relations were then examined and the movements at those periods least affected by tributaries, and best suited to give normal values for different stages, were taken and averaged for the general relation.

This result, expressed in miles per day and taken on the same stage scale in which the $\frac{dQ}{dh}$ of this reach has been given, is

$$m = 70 + 3.27\, h_m.$$

It is thought to be correct to at least within 3 or 4%, but it does not show such limits as distinctly as in the case of the Lower Mississippi. It is the best that could be done with data very imperfectly suited to its determination. Its simple and precise determination in such a river will require other data than those of the ordinary gauge readings.

Equation II, when m is taken in miles per day instead of feet per second, becomes

$$\frac{d\,Q}{d\,h} = \frac{5\,280}{86\,400}\,m\,W,$$

$$\text{or, } W = \frac{16.364}{m}\,\frac{d\,Q}{d\,h},$$

and substituting the values of $\frac{d\,Q}{d\,h}$ and m gives finally, for the equation of the general regimen of the Missouri from Kansas City to St. Charles,

$$W = 16.364\,\frac{1.117\,(h_m + 16.3)^3}{70 + 3.27\,h_m}.$$

This, of course, is simply an equation of dimensions. It gives the value of W from high to low water, and, as such, determines the dimen-

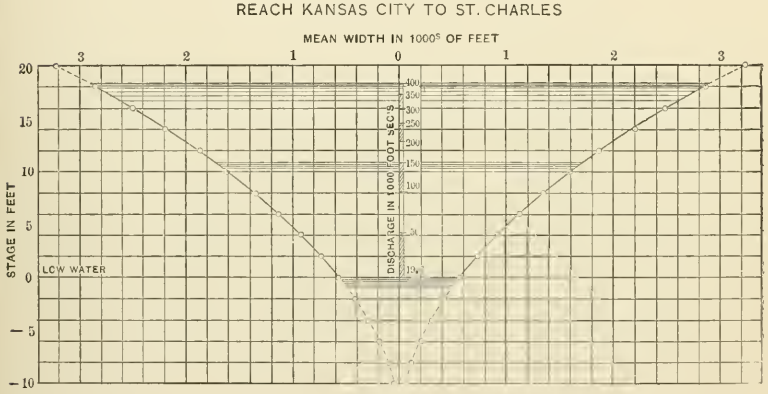

Fig. 14.

sions of the mean hydraulic prism through this reach of river. But no co-ordinates of position have been taken anywhere, and therefore any showing of its mean cross-section must have a further assumption; for any number of areas may be taken between two levels, all having the same value and the same variation, but all differing in their actual forms.

Assuming that W increases symmetrically from an imaginary center line, Fig. 14 shows the mean cross-section of the prism for this reach. Of course, in a divided channel, the actual increase of W with stage, might be altogether toward the center, while at low water especially, its increase would be in any and every direction; but it does not make any difference in what direction it increases, so that the amount and rate of its change is seen distinctly, and this is shown very well in Fig. 14.

Above the water line of the 400 discharge, the river is more or less out of its banks, and that is the limit of this section. What $\frac{dQ}{dh}$ and m may be above that is not known, but it is known that they are not continuous. In the same way there is no guarantee that they would be continuous if they could be observed below low water; but this section is not either of them, but a form of the river, which would be there just the same if all the water on the instant were taken out of it, and this relation has been carried down to 10 ft. below low water, as the most probable continuation of this form as it approached its lower limits.

Between these water lines, however, this regimen of the Missouri is simply measured; not, of course, directly, but as definitely as a distance may be measured with a base and two angles. This measure also has in it probably about the precision of ordinary cross-section data; but it brings out a marked form of bed which all the direct cross-section data has never even suggested. Indeed, it is only in the late surveys on the Lower Mississippi, where the section lines were leveled over all bars and up to the high-water limits, and large means made up from these, that types of this character began to be shown. That the surveys would not give the type accurately is self-evident; for any artificial method of making up the means from them can hardly fail to average values which belong to decidedly different stages. But in the summations of these surveys the essentially triangular character of the Lower Mississippi regimen is well brought out, which its constant flood movements and second-degree discharge curves also give for it.

It is this type of regimen which represents plainly the general equilibrium between the erosive and bar-building forces of the given river; and the type in the Missouri is certainly sufficiently striking. Unlike the plain triangle of the Mississippi, its curvature shows that its forces tend to cut a sharper central thread and build up a widening bar area as the stages are higher. The fact, also noted before, that the low-water level in the Missouri was especially susceptible to the varying forces found in the sequence of high and low-flood years, is here made very plain. Considering the erosion, its action practically consists in cutting down the high banks into this central thread or deeper portion; for the bottom, at the foot of a cutting bank, is rarely above low water. All the caving, therefore, from the cutting banks and bars falls into this central thread and through the low-water season it all stays there,

while the flood period can never fill any of it back above the stage to which the flood reaches.

It is clear that, in the succession of low-flood years, this normal regimen may be changed very materially, this central thread filled up considerably, with all the widths of the lower portion reduced, and the widths of the upper stages increased correspondingly. While again, to maintain the general average regimen of the normal, the action of the succession of high-flood years will be a variation of regimen, from the same causes, but equal and opposite in its character.

Fig. 15, in which the different regimens of 1883 and 1895, before referred to, are given, will serve to show this action. Neither of these regimens has the data needed to determine it absolutely, and they

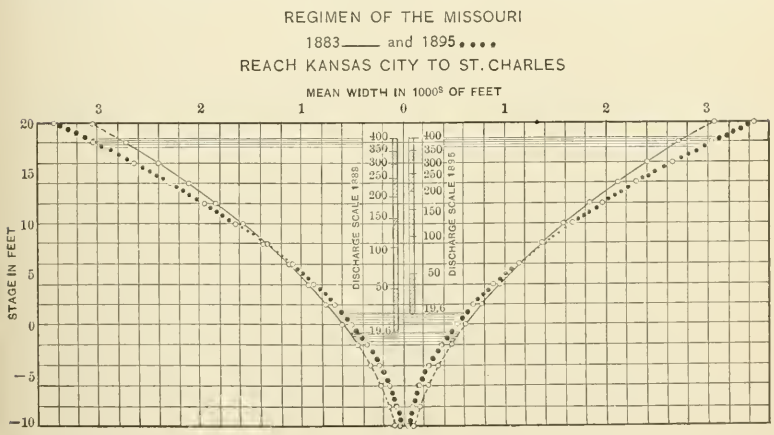

Fig. 15.

should not perhaps be taken as more than illustrations. Thus they are not definite enough to enable one to say with confidence that the extreme range between erosion and filling over this reach is a variation in mean width of some 640 ft. at high water and some 184 ft. at low water. But they are definite enough to indicate that they are something like this, and that they are certainly complementary and always opposite actions of the same system of forces.

This application of Equation II to the determination of the regimen of the Missouri is simply given as an instance of its uses. It is seen that the whole meaning of the term is changed by it; in the place of regimen being but a convenient expression for a sequence of unknown phenomena with no known relations between them, it becomes the re-

cognized resultant of the counter-actions of erosion and bar building, determined accurately and calculated easily.

Of course nothing is calculated easily from data which are not adapted to it, and the laborious processes here followed should never have to be repeated. Continuous automatic-gauge records, referred to the proper plane, should read the flood movement directly between their traces of stage, and give the values of m with precision from tributary to tributary, and through reaches which have no great changes of flood forms. The longitudinal sequence of the equal stages in these reaches is also easier to get than high and low-water slope lines, and, once determined, its mean value for the reach is probably much more nearly a constant. With these then it is simply a matter of taking groups, each of some six or eight discharge observations, at high-water, at mid-stage and at low-water, to determine positively the scales of stage at one or more locations in a long river. And, transferring these from gauge to gauge, the whole river is then known absolutely, and its form and variations of form can be calculated.

It is this which is offered, in the place of the endless soundings of cross-sections, for the study of the effect of work in the construction reaches, levee effects or reservoir effects, and for all the sequence of changing regimens in alluvial rivers, which perhaps no amount of time and money spent in hydrographic surveys would ever even detect.

It is hardly necessary to say that such data may open new problems in river hydraulics; this method only promises to answer those in sight; and even for those more experiment and more work than is shown herein may possibly be needed. But it is worth understanding clearly that the value of $\frac{dQ}{dh}$ and m for every point of stage is the equivalent of a water-line survey of the reach at that stage. True, the actual measures of distance in the survey have a smaller percentage of error than can be looked for in these calculated results, but measuring up an irregular surface area from the field measures has not. The real defect in the survey, however, is that it can never catch the reach at any one stage, and to reduce it to a uniform stage is difficult, to say the least of it. On the other hand, the flood movement not only takes the areas on planes of equal stage, but, in its sequence from high to low water, readily integrates them into the whole river prism.

Of course, with the regimen given, Equation II may be used to cal-

culate the discharge; but in these changing regimens there is always the corresponding uncertainty in a determination of the absolute value of discharge from gauge readings. No surface levels of any kind in any channel can give its discharge unless some element of the size of the channel is given at the same time, and, in rivers like the Missouri, the first thing is certainly to understand its changes of regimen.

But there are other rivers whose regimens may be fixed, or at least which change so little that it would be enough to check them at long intervals or ignore their changes altogether, and in these, when the regimen of a reach had been once established, the flood movement through it would give a continuous record of its discharge, in all its conditions of rise or fall, probably as accurately as if its waters had been run through some of the best studied forms of weirs. And if the drainage system of a continent is ever to be taken, and such of its elements determined as can be put together consistently, it would certainly seem that some such method of measuring it up and keeping its record would be absolutely necessary.

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PAPERS AND DISCUSSIONS.

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EXPERIMENTS ON THE PROTECTION OF STEEL AND ALUMINUM EXPOSED TO WATER.

By A. H. SABIN, Assoc. M. Am. Soc. C. E.

TO BE PRESENTED DECEMBER 20TH, 1899.

This paper states the results of the continuation of a series of experiments to determine the comparative durability of paints and varnishes, which experiments were begun in February, 1895, and were made the subject of a paper read before this Society on November 4th, 1896.* The plates described in that paper had been exposed to the action of sea water in the New York Navy Yard for a period of six months; those to be described now were chiefly triplicate sets, one set of which was exposed under the surface of the sea water in the New York Navy Yard, another was similarly placed in the Norfolk Navy Yard, while the third was placed in fresh water, in Lake Cochituate, near Boston, Mass. All were placed in the water during the last half of June, 1897. Those at Norfolk and Boston remained untouched until July, 1899, a little more than two years; but those in the New York waters were in cages suspended to a float which was accidentally sunk in July, 1898, and more than half the plates were lost; the remainder

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in *Transactions*.

* *Transactions*, Am. Soc. C. E., Vol. xxxvi, p. 483.

were taken out July 21st, 1899, after an immersion of exactly thirteen months.

The consent of the Navy Department was obtained for placing the plates in the Navy Yards, and in this whole matter the author is under great obligations to Naval Constructor F. T. Bowles and Assistant Naval Constructor R. M. Watt, of the New York Yard, and to Naval Constructor A. W. Stahl, of the Norfolk Yard. The Boston set was placed by the courtesy of Desmond FitzGerald, President, Am. Soc. C. E., Engineer of the Sudbury Department, Metropolitan Water Board of Massachusetts.

The assistance rendered by these gentlemen was essential to carrying on the experiments. The frames or cages, containing the plates of steel and aluminum which had been painted, were placed in and removed from the water by their assistants; the New York plates, on removal from the water, were inspected by Mr. Bowles and Mr. Watt, also by representatives of some of the technical journals, and by several engineers; the Norfolk set was examined by Mr. Stahl and by an expert, under his directions. The Engineering Department of the Metropolitan Water Board had a representative on the ground to inspect the plates at Lake Cochituate, but, owing to unexpected difficulties encountered in getting the plates from the water, it was found convenient to make a local inspection.

With the exception of the ships'-bottom paints, which were applied in the New York Navy Yard, the work of preparing and applying the coatings was done by Edward Smith & Company, of New York City, who paid all the expenses incident to the whole work. The aluminum plates were furnished free of charge by the Pittsburg Reduction Company.

The plates were suspended in open cages or racks, and were in a horizontal position one above another, about 2 ins. apart. They fitted tightly in the racks, and each plate was supported at the four corners. The racks were suspended about 6 ft. below the surface of the water, but at Lake Cochituate they were laid on the bottom, which was hard, about 20 ft. below the surface, and the plates were vertical. Besides the plates lost at New York, one rack containing 25 plates was lost at Norfolk. It was suspended by two $\frac{3}{8}$-in. galvanized iron chains, which rusted off. This gives some idea of the severity of the exposure. All the Boston plates were recovered.

In the salt-water tests it was found that barnacles and other organisms attached themselves to the lower sides of the plates. In the Norfolk Navy Yard the action of these was so severe as to destroy the paint on the under sides of all the plates, with the exception of those coated with the Sabin Pipe Coating, which did not seem to be affected, although oysters 4½ ins. in length were found growing on it. When these were removed the coating was found to be intact. But with this exception, it should be remembered, in looking over Table No. 1, that only one side of the plates in the Norfolk set is described, the coatings on the other side being uniformly destroyed, while in the New York and Boston sets both sides of the plates are included in the description.

Such varnishes as are mentioned in this paper are made by melting, at a temperature of 700° to 900° Fahr., certain resinous substances, such as Kauri, Manila and Zanzibar resins, and the asphaltic mineral known as gilsonite; 100 lbs. of resin is usually melted at once, and to this is then added a quantity of linseed oil, also hot, the amount of which varies according to the kind of varnish to be made. If 20 galls. of oil is used the product is called a 20-gall. varnish; if 30 galls., it is a 30-gall. varnish; and so on. The mixture of resin and oil is then heated for some hours until combination occurs, and the product is then thinned with spirit of turpentine, which is regarded as a vehicle. Varnishes made with 20 galls. of oil, and with Manila, Kauri and Zanzibar resins, would be designated as 20 M., 20 K., and 20 Z., respectively; and so on. Enamel paints are made by grinding pigments in these varnishes, exactly as oil paints are made by grinding the same pigments in linseed oil.

Inasmuch as it has been believed that the process of baking adds to the durability of these coatings, in a few cases duplicates were prepared, one of which was baked and the other allowed to dry at the ordinary temperature.

The general scheme was to apply to a set of four plates a set of three varnishes, 20 K., 30 K. and 40 K., and pure raw linseed oil. Then, for another similar set, these same liquids were mixed, by grinding with white zinc; another set was prepared with white lead; another set with ultramarine blue; another set with graphite; and so on. This ought to show whether one pigment is better than another, and which vehicle is the best. Besides these, plates were painted with

pure red lead in pure linseed oil, with two mixtures of red lead and white zinc, with purple oxide of iron (crocus) in oil, and with Prince's metallic oxide of iron, which is a very well-known pigment consisting of iron oxide mixed with various silicates, in oil.

Besides these coatings of known composition, two popular and widely-known proprietary paints, the Eureka paint and the graphite paint made by the Detroit Graphite Manufacturing Company, were tried. These oil paints and proprietary paints were presumed to afford a sort of standard by which the other coatings could be judged.

The coating material described in the table as "Spar," is one of the well-known class of spar varnishes used for exterior work, and the kind used was made by Edward Smith & Company. The "I. X. L. No. 2" is a well-known interior varnish. The substance indicated by the initials "D. M. C." is Edward Smith & Company's "Durable Metal Coating," which is a varnish in which a considerable amount of gilsonite has been substituted for a corresponding quantity of resin. "Parahydric" is a coating similar to "Durable Metal Coating," but containing less oil, and has been used in painting the interior of water-pipes. "S. P. C." is the Sabin Pipe Coating, a varnish enamel which is baked on at about 400° Fahr., and which has been used on some of the largest hydraulic works in this country and is largely used in coating pipes on United States naval vessels. "Keystone" is a well-known pigment, probably ground slate, and was used to furnish a pigment composed of silicates for comparison. The iron oxide used is the purest commercial sesquioxide, containing over 95% oxide of iron. The purple oxide of iron is oxide which has been subjected to prolonged heating, and is supposed to be anhydrous. The "iron oxide in shellac" mixture was prepared from a formula furnished by Naval Constructor Bowles; the shellac is pure "D. C. Shellac" in grain alcohol. The paints known as Rahtjen's, McInnes' and Holtzapfel, are anti-corrosive and anti-fouling ships'-bottom paints, and were furnished and applied by the New York Navy Yard.

The nomenclature and abbreviations of Table No. 1 are as follows:

20 K. = 20-gall. Kauri Varnish.
30 K. = 30 " " "
40 K. = 40 " " "
20 M. = 20 " Manila. "

30 M. = 30-gall. Manila Varnish.
40 M. = 40 " " "
20 Z. = 20 " Zanzibar "
30 Z. = 30 " " "
40 Z. = 40 " " "
D. M. C. = Durable Metal Coating.
S. P. C. = Sabin Pipe Coating.
Um. Blue. = Ultramarine Blue.
W. Z. = White Zinc.
W. L. = White Lead.
A. = Pure Aluminum.
A. A. = Aluminum Alloy, 95 per cent.

The plates used for the purpose were of uniform size, 12 x 20 ins.; part of them were aluminum and the remainder steel, about $\frac{1}{8}$ in. in thickness. Since it is well known that aluminum is not acted on by fresh water, the aluminum plates were all put in the New York and Norfolk sets. The plates numbered from 151 to 163, inclusive, in the New York set, correspond to those numbered from 176 to 187, inclusive, in the Norfolk set, and are pure aluminum; those numbered from 164 to 175, New York, correspond to 188 to 200, Norfolk, and are aluminum plates alloyed with 5% of other metal.

Besides these regular sets of plates, a cage containing 24 plates, part steel and part aluminum, which had in 1896 been exposed for six months in the New York Yard, and are described in the author's paper in November, 1896, were again exposed with the New York set. Half of these were lost by the accident already referred to, but the remainder are described in Table No. 1. Of these, Nos. 104, 105, 113, 122 and 124 are aluminum,* and the rest are steel. All are distinguished in the table by the date (1896) after the number.

As two entire cages of steel plates were lost in the New York Yard and one in the Norfolk Yard, the descriptions of the aluminum and the "1896" plates have been put in the otherwise vacant spaces of the table. All the plates, except those coated with Sabin Pipe Coating, which had two coats, received three full coats, well dried between coats. The red-lead paint used weighed about 35 lbs. to the gallon, and was put on with the plate in a horizontal position, on the top side of the plate. After the paint had set, the plate was turned over and

* See paper already referred to.

the other side was painted. The steel plates were all of the grade known as pickled and cold-rolled, and were cleaned with a steel wire brush. They were not pickled after they had left the steel mill, but their surfaces were perfectly clean and bright after being brushed.

Results.

In most cases, the protective coatings seem to begin to deteriorate on the surface, which becomes rough; then little blisters appear, which are caused by the separation of the last coat from those beneath; finally the undercoat blisters, in which case it is found, almost invariably, that rust has formed under the blister. If, however, the coating is porous—and this seems to be the case with the ordinary oil paints—the water reaches the metal and causes rust. This throws off the paint film, and the corrosion spreads rapidly in this way.

Protective coatings, such as are applied with a brush, are of three classes:

1. Pigment paints, in which the vehicle or liquid part is either linseed oil (or an inferior substitute) or varnish, and with this liquid portion is mixed mechanically a powdered solid, usually a mineral substance ground to a fine powder;

2. Red lead, which is a mixture of red lead and linseed oil, and which does not dry by oxidation as do other oil paints, but sets like a cement; and

3. Varnishes.

The linseed oil in ordinary oil paints dries by the absorption of oxygen to a tough leathery substance, but the film thus formed is soft and gummy, and is quite porous. Pigment is added to the oil for three reasons. It makes the liquid thicker and more viscous, thus enabling the operator to put on a thicker film; it makes a harder film, which will better resist abrasion; and the particles of pigment tend to fill up the pores of the film, thus rendering it more continuous.

Varnish forms a much more continuous (less porous) film than oil, hence it is better adapted for use in paint, and this is shown conspicuously by an examination of these tests, where the oil paints have failed, without exception, while the corresponding varnish paints are, in most cases, in good condition. The character of the pigment does not seem to have much influence. All the oil paint samples were so badly rusted that differentiation among them was impossible. It may

be that an earlier inspection would have shown differences, but the present appearance of all these plates is so similar that it seems unlikely, and certainly the varnish paints do not show any great differences in the matter of the pigments, except that white zinc seems to be somewhat the best. The iron oxides, graphites, and pulverized slate are all alike. The red lead, in the Boston and New York sets, is far better than any of the oil paints. The mixtures of red lead and white zinc are markedly inferior to red lead alone. In the Norfolk test, which was much more severe, the red lead had finally been quite destroyed. Deterioration in the case of red lead seems always to start from centers. In the Boston set, the red lead seemed to be in pretty good condition, but would probably not have lasted more than a year or so longer. It shows numerous rust spots. The blisters all start from the metal. But most of the varnish paints are much better than the red lead.

A study of the varnishes applied without pigment seems to show that in the fresh-water exposure the process of baking was, on the whole, of advantage, but not greatly so. In the salt water, the unbaked varnishes were better than the same varnishes baked; this agrees with the results recorded in 1896. The Manila varnishes are distinctly inferior to the Kauri and Zanzibar; the "Durable Metal Coating" is best of all. This is doubtless due, in a large degree, to the fact that this varnish, which is intended especially for the protection of structural steel, is made with a heavy body, and the film is of greater thickness than is the case with varnishes intended for wood work. Its composition has also been very carefully studied and designed to secure great durability, which is of much less importance than other qualities in ordinary varnishes.

By far the best results, however, with the exceptions to be hereafter noted, were obtained from the best of the enamel paints. Here, also, the Manila varnishes are decidedly inferior, and, in the opinion of the author, these should be excluded hereafter from any such tests, although they make a much better comparative showing on wood.

In the enamel or varnish paints, those made with the more elastic varnishes (those containing the most oil) are decidedly the better. The extreme durability of these is well shown by the "1896" plates. These were first exposed to the air two or three months, then they were in the sea water six months, then exposed to the air nearly a year,

then under water thirteen months, and have since been exposed to the air over sixteen months, making a total of four years, and they are still, to all intents, perfect.

It is true that the air exposures have been indoors, but it was in a building adjacent to a railroad yard, where unprotected iron and even nickel steel is covered with deep rust in two or three months, and the air is often so acrid as to cause coughing. Two years' continuous immersion in fresh water has not injured some of these enamels, and two years in the excessively severe exposure at the Norfolk Navy Yard has left several of them in good condition, a few being practically uninjured.

It is generally true of all the better class of coatings that corrosion begins at the edges of the plate. In the case of aluminum plates, it seems evident to the author that some of these coatings, notably the "Spar varnish" and the "Durable Metal Coating" have been gradually thrown off by corrosion creeping from the edge, probably from some mechanical injury, under the varnish, a patch of which remains uninjured and apparently without deterioration on the middle of the plate. This fact should not be lost sight of in considering this matter, and is one of the points shown by an inspection of the plates, but not easily brought out in a description.

Undoubtedly, the most obvious and conspicuous, and the most instructive, fact, is the total and absolutely universal failure of linseed oil films, either alone or mixed with any of the numerous pigments which were tried, while the corresponding varnishes and enamel paints made with the same pigments are in fair to good condition. This confirms strikingly the opinion, long held by the author and many others of greater experience, that varnish films are much more impervious and resistant than any others. The exceptional cases to be noted are :

First.—The "Sabin Pipe Coating," a baked enamel, which is so much superior to the others as to form a class by itself, and

Second.—The extraordinary showing made by pure shellac varnish in the Lake Cochituate test. Shellac varnish is simply a solution of shellac resin, which is chemically an acid substance, in alcohol.

There are many grades of shellac ; the one used was what has for many years been known as "D. C." orange shellac, and it was dissolved in pure 97% grain alcohol. Being an acid substance, it is attacked and dissolved readily by the ammonia in the atmosphere. It

is removed easily by soap and water. It has never been considered a durable varnish as ordinarily used on wood work, and it does not stand at all in the sea-water tests, but two years' exposure under 20 ft. of fresh water does not seem to have injured it sensibly. This may be a very serious matter, for, while in this regard it is no better than some other varnishes, such as "Durable Metal Coating," which cost much less money, shellac varnish has some important and exceedingly desirable qualities, which no other varnish has. For example, occasionally we are confronted with the problem of repainting a section of large water pipe which can be spared from use only a few days. The interior of this pipe is damp. The best that can be done with it is to get out most of the visible water, but the cold surface of the metal will always be damp. No ordinary varnish will stick to such a surface, and corrosion will probably be set up at once; no ordinary varnish, of sufficient durability to be worth putting on, will dry in the limited time at our disposal. Now, shellac is dissolved in a vehicle which has an intense affinity for water, and it is highly probable that a thin film of dew will be instantly absorbed, and will certainly be removed, by the evaporation of the slightly diluted alcohol; and shellac, if applied in a thin coat, dries with the greatest rapidity. Three coats may be applied in eight hours. There is no unpleasant or dangerous odor, though ventilation should be secured, both on account of the risk of fire and because working in an atmosphere of alcoholic vapor produces intoxication. It certainly seems from this test as though we would be justified in using shellac varnish in such a case. It is expensive, of course, and it is very likely that the cheaper grades, which are found in ordinary use to be very much inferior in durability, would not be so efficient. In any case it would not be necessary to use it where the conditions are such that some equally good but slower drying coating can be used.

The metal plates used in these tests were free from rust or scale, and showed in all cases the bright color of the metal. To prepare plates for a paint test, this is the only way which will give comparative results. The author is positive about this. It is sometimes said that we do not get our metal in this condition when we paint it, and therefore it is proper to make tests on somewhat rusty iron, or on iron with mill scale on it. In the first place, we ought to get the iron in that clean condition, and the author is not without hope of liv-

ing to see the day when it will be done, in the case of permanent structures; and secondly, he is satisfied, from many years' constant experience in such experiments conducted on a most extensive scale, that it is absolutely impossible to get uniform conditions with rusty metal. A dozen metal plates, even if cut from the same sheet, will not rust uniformly; one will throw off a scale and another will not. The mill scale on a plate will be almost entirely removed from part of a plate by passing it through bending rolls, while it will adhere firmly to another part of the same plate. If this plate had been cut up and used for tests, the paint on one piece would come off and on another would stay on, and the scale is always different on the two sides of a plate.

Exposure tests, such as these, are of much more importance than laboratory tests. The manufacturers of paints and varnishes, some of whom are probably the best experts in this matter, never depend on any but an exposure test. It is by no means impossible that rapid laboratory tests may yet be devised, but such crude ones as have been so far proposed are in most cases of little value. Such a test, for example, is that with caustic alkali. This is a substance unknown in Nature, and no good paint will stand it, while a perfectly worthless paint might be made which would stand it very well. A nitric-acid test is of the same sort. It will simply burn up any organic substance, and some of the best linseed oil paints will yield to it most readily. It would hardly be regarded as a fair test of the comparative health of a dozen human beings to administer to each of them a couple of ounces of nitric acid and keep watch to see which lived longest, yet probably each could take a few drops of it per day without inconvenience. This is about what many of these so-called paint tests amount to. Some laboratory tests are of considerable value, but none is conclusive.

It has frequently been objected to these submarine tests that they are of value only as regards the same conditions, and there is some justice in such a criticism; but it is much weakened by the obvious fact that there is a practical agreement between the fresh-water and the sea-water tests. The latter were most severe, but in most cases the difference has been one of degree only. And in the rather large experience of the author and his associates, these tests seem to agree in general with aerial exposures, reasonable exception being made in the case of coatings intended expressly for marine or for aerial use.

TABLE No. 1.

Lake Cochituate, Boston.	401-20 K. No rust except where damaged along edges; many very small blisters.	404-20 M. Much rust; coating much injured.	407-20 Z. Not much corrosion, but coating about destroyed.
	402-30 K. Like 401, not quite so good.	405-30 M. Worse than 404; coating nearly destroyed.	408-30 Z. Like 407, but considerably better.
	403-40 K. Like 401.	406-40 M. Not quite so bad as 405.	409-Spar. Like 408, but perhaps a little better.
Navy Yard, Norfolk, Va.	301-20 K.	304-20 M.	307-20 Z.
	301 to 310, coatings not destroyed; all considerably injured; blistered in small spots; no considerable corrosion; 301 worst; 306 and 309 best. 307-8 not bad.		
	302-30K.	305-30 M.	308-30 Z.
	303-40 K.	306-40 M.	309-Spar.
Navy Yard, New York.	1 (1896)-W. L. in 20 K. Some rust along edges; otherwise in good condition.	16 (1896)-W. Z. in 20 M. One-fifth of one side rusted; all the rest in good condition.	47 (1896)-Spar. Coating firm and good; very little rust.
	2 (1896)-W. L. in 20 K. Like 1.	18 (1896)-W. L. in 8 M. Paint hard and firm; in good condition.	35 (1896)-S. P. C. A little corrosion near the edges; otherwise all right.
		20 (1896)-W. Z. in 30 Z. Good. No blisters; no rust.	113 (1896)-S. P. C. Two small blisters; otherwise good.
		22 (1896)-W. Z. in 20 Z. Good. Some corrosion along edges.	

. TABLE NO. 1—(*Continued*).

410–I. X. L. No. 2. About like 407.	413–D. M. C. Good, except where broken and injured along edges.	417–Parahydric. Numerous isolated rust spots about ¼ in. diameter; coating otherwise good.	Lake Cochituate, Boston.
411–Shellac. Very excellent condition.	414–D. M. C. Like 413.	418–Parahydric. Like 417.	
412–Raw Oil. Surface generally corroded; many tubercles.	415–D. M. C. Like 413.	419–Parahydric. Like 417.	
	416–D. M. C. Like 413.	420–Parahydric. Like 417.	
310–I. X. L. No. 2.	313–D. M. C. Many small blisters, in outer coat chiefly; very little corrosion.	317–Parahydric. Coating all on; no blisters.	Navy Yard, Norfolk, Va.
311–Shellac. Coating practically gone; badly rusted.	314–D. M. C. Like 313.	318–Parahydric. Like 317.	
312–Raw Oil. Coating destroyed; very badly rusted.	315–D. M. C. Like 313.	319–Parahydric. Like 317.	
	316–D. M. C. Like 313.	320–Parahydric. Like 317.	
124 (1896)–Spar, one side baked. Very few small blisters, otherwise perfectly good.	105 (1896)–Chromium oxide in 20 K., one side baked. A few blisters; otherwise in excellent condition.		Navy Yard, New York.
104 (1896)–W. Z. in 20 K., one side baked. Like 124.			
122 (1896)–W. Z. in 20 K., one side baked. Like 122.			

TABLE No. 1—(*Continued*)

Lake Cochituate, Boston.	421-S. P. C. Perfect, except where coating is in one or two places broken at edge with corrosion.	425-W. Z. in 20 K. Half the surface, along the edges, blistered, with rust underneath.	428-W. Z. in 20 M. Outer layer of coating nearly destroyed; under coat good.
	422-S. P. C. Like 421.	426-W. Z. in 30 K. Much better than 425; some blisters; little corrosion.	429-W. Z. in 30 M. A few slight rust spots; outer coat blistered.
	423-S. P. C. Like 421.	427-W. Z. in 40 K. Good condition; some blisters in outer layer of coating; no rust.	430-W. Z. in 40 M. About one-fifth rusted; thin rust. Blistered; outer coat chiefly.
	424-S. P. C. Like 421.		
Navy Yard, Norfolk, Va.	321-S. P. C. Perfectly good condition. See note in text.	325-W. Z. in 20 K. Blistered; not very good.	179 A-I. X. L. No. 2. Coating all gone.
	322-S. P. C. Like 321.	176 A-20 K. Three-fourths of coating destroyed; thin rust.	180 A-Spar. Two-thirds of coating gone, but one-third in the middle perfectly good.
	323-S. P. C. Like 321.	177 A-30 K. Like 176.	181 A-D. M. C. One-fifth gone, one-fifth blistered; remainder good.
	324-S. P. C. Like 321.	178 A-40 K. Coating all gone.	182 A-S. P. C. One-tenth gone on one edge; remainder all right.
Navy Yard, New York.			154 A-I. X. L. No. 2. Varnish half gone. Corrosion not deep.
		151 A-20 K. Blistered along edges and a few spots. Varnish firm. Little corrosion.	155 A-Spar. Most of the varnish soft, but some not affected. Not badly corroded.
		152 A-30 K. Much corrosion; some deep. Coating half gone; remainder firm.	157 A-D. M. C. Twenty per cent. blistered around edges. Coating firm; not much corrosion.
		153 A-40 K. Badly corroded; coating nearly all destroyed.	158 A-S. P. C. Excellent. Coating not injured, except by accident in removing from frame.

TABLE No. 1—(*Continued*).

431–W. Z. in 20 Z. Not much rust; outer coat badly blistered; under coat slightly so.	436–W. Z. in 20 K., baked. Almost perfect; still shows glossy surface of varnish.	439–W. Z. in 20 M., baked. Good; coating brittle in places and shows deterioration.	Lake Cochituate, Boston.
432–W. Z. in 30 Z. Better than 431. Outer coat blistered.	437–W. Z. in 30 K., baked. Like 436.	440–W. Z. in 30 M., baked. A little better than 439.	
433–W. Z. in Spar. Like 432.	438–W. Z. in 40 K., baked. Like 436.	441–W. Z. in 40 M., baked. Almost perfect.	
435–W. Z. in Raw Oil. Four-fifths of surface badly rusted; deep corrosion.			
183 A–S. P. C. Perfectly good condition.	187 A–W. Z. in Spar. Like 184.	191 AA–I. X. L. No. 2. Coating all gone.	Navy Yard, Norfolk, Va.
184 A–W. Z. in 30 K. Fine; no rusting nor blistering.	188 AA–20 K. Coating all gone.	192 AA–Spar. Three-quarters gone; like 189.	
185 A–W. Z. in 40 K. Like 184, but discolored.	189 AA–30 K. Three-quarters gone; small patch in the middle all right.	193 AA–Spar. Like 192.	
186 A–W. Z. in 30 Z. Like 184.	190 AA–40 K. Half gone; like 189.	194 AA–D. M. C. One-third badly blistered from edges; remainder good.	
159 A–S. P. C. Like 158 A.	163 A–W. Z. in Spar. Not deeply corroded. Several large blisters: otherwise in good condition.	167 AA–I. X. L. No. 2. Considerable blistering and corrosion. Coating easily scraped off.	Navy Yard, New York.
160 A–W. Z. in 30 K. Upper side perfect; lower side slightly blistered. Coating hard.	164 AA–20 K. Badly corroded; three-fourths of the varnish destroyed.	168 AA–Spar. Like 167, but not badly corroded.	
161 A–W. Z. in 40 K. No blisters; otherwise like 160 A.	165 AA–30 K. Like 164 AA.		
162 A–W. Z. in 30 Z. Like 161 A.	166 AA–40 K. Badly blistered, but not badly corroded. Coating on one side firm; on the other soft.	169 AA–D. M. C. Many blisters: very little corrosion; coating generally firm.	

TABLE No. 1—(*Continued*).

Lake Cochituate, Boston.	442–W. Z. in 20 Z. baked. Nearly perfect.	445–W. L. in 20 K. Very little corrosion. Some superficial blisters.	449–Um. Blue in 20 K. Considerable rust; not deep; paint practically destroyed.
	443–W. Z. in 30 Z. baked. Excellent; no rust; blisters superficial and few.	446–W. L. in 30 K. Good condition; no rust. Some superficial blisters.	450 Um. Blue in 30 K. A little worse than 449.
	444–W. Z. in Spar, baked. Like 443, or better.	447–W. L. in 40 K. Like 446.	451–Um. Blue in 40 K. Worse than 449; deep rust.
		448–W. L. in Raw Oil. Much deep corrosion; about half the plate in good condition.	452–Um Blue in Raw Oil. Like 451; whole surface rusted.
Navy Yard, Norfolk, Va.	195 AA–W. Z. in 30 K. Good; blistered a little on the edges.	199 A A–S. P. C. Blistered a little from edges; otherwise all right.	
	196 AA–W. Z. in 40 K. Fine, but discolored; like 185.	200 AA–S. P. C. Like 199.	
	197 AA–W. Z. in 30 Z. Fine, but blistered a little along the edges.		351–Um. Blue in 40 K. Nearly all gone.
	198 AA–W. Z. in Spar. Like 197.		352–Um. Blue in Raw Oil. Coating all gone; very badly rusted.
Navy Yard, New York.	170 AA–W. Z. in 30 K. Very little corrosion. Blisters amount to 1%. Coating good.	174 AA–S. P. C. In perfectly good condition.	
	171 AA–W. Z. in 40 K. Good, but not equal to 170 AA.	175 AA–S. P. C. Like 174.	
	172 AA–W. Z. in 30 Z. No corrosion; no blisters; excellent condition.		251–Um. Blue in 40 K. Very many small blisters; very little corrosion.
	173 AA–W. Z. in Spar. About like 172.		252–Um. Blue in Raw Oil. Uniformly corroded; coating all gone.

TABLE No. 1—(*Continued*).

453–Graphite in 20 K. Very good; some small blisters.	457–Keystone in 20 K. Good condition; no rust; some small blisters.	461–Iron Oxide in 20 K. Very little rust; small blisters in outer coat	Lake Cochituate, Boston.
454–Graphite in 30 K. Like 453.	458–Keystone in 30 K. Like 457.	462–Iron Oxide in 30 K. Better than 461; no rust.	
450–Graphite in 40 K. Like 453.	459–Keystone in 40 K. A little rust; many small superficial blisters.	463–Iron Oxide in 40 K. Like 462.	
455–Graphite in Raw Oil. Deeply and generally rusted; about one tenth of the paint still good.	460–Keystone in Raw Oil. Badly and deeply rusted; patches of paint still good.	464–Iron Oxide in Raw Oil Corrosion deep and general; paint all gone.	
353–Graphite in 20 K. Three-quarters gone; much rust.	357–Keystone in 20 K. Coating blistered and one-quarter gone.	361–Iron Oxide in 20 K. Pretty good condition; a few blisters.	Navy Yard, Norfolk, Va.
354–Graphite in 30 K. Half gone; much rust.	358–Keystone in 30 K. Blistered, but not destroyed	362–Iron Oxide in 30 K. Not quite as good as 361.	
355–Graphite in 40 K. One-quarter gone.	359–Keystone in 40 K. Blistered, but not in bad condition.	363–Iron Oxide in 40 K. Like 361.	
356–Graphite in Raw Oil. Nearly all gone; badly rusted.	360–Keystone in Raw Oil. All gone; badly rusted.	364–Iron Oxide in Raw Oil Like 360.	
253–Graphite in 20 K. A few blisters; very little corrosion.	257–Keystone in 20 K. No corrosion; numerous very small blisters.	261–Iron Oxide in 20 K. Blistered, but not very badly. Not much corrosion.	Navy Yard, New York.
254–Graphite in 30 K. Like 253.	258–Keystone in 30 K. Like 257.	262–Iron Oxide in 30 K. Like 261. Not deeply rusted.	
255–Graphite in 40 K. No corrosion. Paint in good condition. Numerous very small blisters.	259–Keystone in 40 K. Like 257.	263–Iron Oxide in 40 K. Like 262.	
256–Graphite in Raw Oil. Uniformly corroded; coating all gone.	260–Keystone in Raw Oil. Coating destroyed and plate badly corroded.	264–Iron Oxide in Raw Oil. Like 260.	

TABLE No. 1—(*Continued*).

Lake Cochituate, Boston.	465–Red Lead in Raw Oil. Paint still tough; looks well. Blisters from the bottom with slight corrosion beneath.	469–Eureka Paint. General corrosion: paint entirely destroyed.	
		470–Detroit Graphite. Like 469; paint nearly all destroyed.	475–International Holtzapfel. Like 469.
	467–Prince's Metallic in Raw Oil. About one-quarter deeply rusted; paint practically all gone.	472–Iron Oxide in Shellac mixture. Good condition; about 2% rusted.	477–Red Lead and W. Z. in Raw Oil. Many deep rust spots; about 5%; remainder good.
	468–Purple Oxide in Raw Oil. Like 467.		478–Red Lead and W. Z. in Raw Oil. Like 477. Not nearly as good as 465.
Navy Yard, Norfolk, Va.	365–Red Lead in Raw Oil. Coating destroyed; plate badly rusted.	369–Eureka Paint. Like 365.	374–McInnes' Paint. Like 372.
		370–Detroit Graphite. Like 365.	375–International Holtzapfel. Like 365.
	367–Prince's Metallic in Raw Oil. Like 365.	372–Iron Oxide in Shellac mixture. Paint destroyed; general, but not deep corrosion.	377–Red Lead and W. Z. in Raw Oil. Like 365.
	368–Purple Oxide in Raw Oil. Like 365.	373–Rahtjen's Paint. Like 365.	378–Red Lead and W. Z. in Raw Oil. Like 365.
Navy Yard, New York.	265–Red Lead in Raw Oil. Coating badly destroyed. Considerable corrosion.	269–Eureka Paint. Like 260.	274–McInnes' Paint. In good condition; no barnacles.
		270–Detroit Graphite. Like 260.	275–International Holtzapfel. Paint badly gone; much corrosion; many small barnacles.
	267–Prince's Metallic in Raw Oil. Like 260.	272–Iron Oxide in Shellac mixture. A few blisters; otherwise in good condition.	277–Red Lead and W. Z. in Raw Oil. Coating thin; gone in many places; considerable corrosion.
	268–Purple Oxide in Raw Oil. Like 260.	273–Rahtjen's Paint. Paint badly gone; considerable rusting. Many small barnacles.	278–Red Lead and W. Z. in Raw Oil. Like 277.

TABLE No. 1—(*Concluded*).

481–20 K., baked. Practically perfect; coating still glossy.	484–30 M., baked. Several deep spots of rust, coating badly blistered.	487–I. X. L. No. 2, baked. Like 481.	Lake Cochituate, Boston.
482–30 K., baked. Like 481.	485–30 Z., baked. Like 481.	488–Raw Oil, baked. Badly and deeply rusted. Two-fifths of the surface good.	
483–40 K., baked. Like 481.	486–Spar, baked, Like 481.	489–D. M. C., baked. Fine; a few small blisters in the outer coat.	
381–20 K., baked. Half of the coating destroyed; the rest good. Not much rust.	384–30 M., baked. Like 382.	387–I. X. L. No. 2, baked. Like 386.	Navy Yard, Norfolk, Va.
382–30 K., baked. Four-fifths destroyed; very little rust.	385–30 Z., baked. Like 382.	388–Raw Oil, baked. All gone; rusted.	
383–40 K., baked. Like 382.	386–Spar, baked. Nearly all gone; little rust.	389–D. M. C., baked. Three-quarters gone; remainder good; very little rust.	
281–20 K., baked. Plate thinly rusted along the edges.	284–30 M., baked. Many small and some medium-sized blisters. Not badly rusted.	287–I. X. L. No. 2, baked. Not much corrosion; very small blisters.	Navy Yard, New York.
282–30 K., baked. Small blisters, with thin rust beneath, over most of the plate.	285–30 Z., baked. Good condition. Very little rusting. Very small blisters.	288–Raw Oil, baked. Badly corroded. Coating destroyed.	
283–40 K., baked. Very small blisters; not much rust.	286–Spar, baked. Coating badly destroyed; much corrosion.	289–D. M. C., baked. Very many small blisters. Not very much corrosion.	

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

ON THE THEORY OF CONCRETE.

Discussion.*

By Messrs. William B. Fuller, G. L. Christian and Alfred F. Harley.

Mr. Fuller. William B. Fuller, M. Am. Soc. C. E. (by letter).—There is one point which, it seems to the writer, has not been made sufficiently clear in this discussion, although it is really the basis of the most economical mixture for any given quantity of cement. If we make a concrete by using merely cement and sand, or cement and stone, it will take a large amount of cement to fill the voids in the sand, or in the stone, for any given quantity of concrete; but if we make a concrete of cement, sand and stone, the sand fills the larger voids in the stone and the cement has only to fill the remaining voids in the mass; a very much smaller percentage than in either of the first two cases. If, in the mixture of cement, sand and stone, the sand and stone are not in proper proportions, the cement is called upon to do more work than it should; that is, it has to fill the larger voids of this excess material. Take, for instance, sand having 45% voids, and stone having 40% voids. With the sand just filling the voids of the stone, it is easily calculated that the resultant mass has 18% voids; but supposing an excess of 10% of sand, there would be 10% of the material having 45% voids, which means there would be 2.5% more voids in the resultant mass.

The 2.5% of cement necessary to fill these extra voids, it is seen, is really wasted, for the resultant concrete would have been much

* Continued from September, 1899, *Proceedings*.

stronger if the sand had been decreased and the stone increased, leaving the same amount of cement 2.5% in excess in the mass. Mr. Fuller.

The above example indicates what frequently happens in mixing concrete. Indeed, it is what is frequently advocated in discussion and by writings on the subject; that is, to make up a mortar of cement and sand and mix this mortar with the stone, being sure that the mortar is slightly in excess, so as to fill all the voids of the stone.

This is what is known as drowning the stone, and the resultant concrete is hardly any better, so far as its properties are concerned, than a concrete of cement and sand alone. After the stone is once drowned no addition of mortar will improve the concrete.

The writer has frequently made a rich concrete of a poor one by simply rearranging the proportions of sand and stone without in the least changing the proportions of cement in the mass.

It must be apparent that there is just one proportion and only one in which any two such materials as sand and stone can be combined and give a minimum number of voids in the resultant mass, and it seems to the writer that this is the proportion in which they should always be mixed, whether we want a poor or a rich concrete. Mix the sand and the stone in a definite fixed quantity as determined by experiment for the particular material in use, and then add cement as economy dictates, possibly up to 10% in excess of the voids of the combined material.

This suggests another proposition which must be almost self-evident, namely, that the proper selection and grading of the sand and aggregate so as to produce the least number of voids will allow of a corresponding reduction in the amount of cement used without reducing the quality of the concrete desired. In making up such mixtures, however, the necessary relation of the size of the particles of the different materials should be kept in mind, that is, the particles of each material should be small enough to easily enter the voids formed by the particles of the next larger material; otherwise they will be left out and themselves form voids, which must be filled by the finer material or remain unfilled, to the detriment of the concrete.

It is quite common to find sand used which is too fine for the cement, or sand in which the larger particles act merely as stone, or stone with its voids partly filled with sand or dust.

The proper proportions of sand and stone can be determined by mixing sample batches and determining their voids by any of the well-known methods; or, another very satisfactory method, as it also gives information as to the relative sizes of the materials, is to make a mechanical analysis of all the materials in the manner prescribed by the Massachusetts State Board of Health for the analysis of sands, and from the resultant curves figure the composition giving the highest

Mr. Fuller. uniformity coefficient. The writer has used the foregoing methods in concrete structures with uniform success in the reduction of the amount of cement required, without reducing the apparent quality, and, by changes in the proportions of the aggregates, has maintained a uniform quality of concrete, through the use of a wide range of aggregates, without changing the percentage of cement.

In discussing the relative merits of hand-mixed and machine-mixed concretes it is quite essential that the kind of machine with which the comparison is made should be stated, as there is as much difference between the products of different machines as between that of good and poor hand-mixing. With a machine of the cubical-box type, into which all the materials, including the water, can be measured accurately, the product, as far as observation can establish it, is superior to the best hand work. With such a machine the writer has seen concrete, of almost absolutely unvarying quality and consistency, produced for days at a time.

Mr. Christian. G. L. CHRISTIAN, Assoc. M. Am. Soc. C. E. (by letter).—The writer has had considerable experience with hand-made concrete, but none with machine-made.

The hand-made concrete, to be of any use, requires continual, hard and conscientious work on the part of the mixers, and continual inspection by an intelligent, live and honest inspector, who will always insist on having everything done well.

The instant the inspector turns his back, the temptation to turn the water on the mortar, before the sand and cement are thoroughly mixed, and, in fact, before they are mixed at all, is too much for the average laborer to resist, especially if he is hard pushed and the day is hot. Taking all things into consideration, the writer would expect to find machine-made concrete superior to hand-made in a large majority of cases.

Only in cases where concrete is exposed to the sun does the writer agree with Mr. Gould as to the advisability of keeping concrete moist for a long time after being placed in the work. The writer has reached this conclusion, partly as a result of a test of quite a number of briquettes which were mixed about five years ago and recently broken. Some were of neat cement, some 2 to 1 and some 3 to 1. One set of each of these was mixed with fresh water, one with 3% salt water, and the other with 13% salt water. Half of each set of briquettes was laid away on wooden shelves in a warm office soon after mixing, and the other half was placed in water and kept there until ready to be broken. Many of these briquettes, have been broken, and, so far, those of whatever kind which were kept dry have shown the greatest tensile strength.

The writer was at one time a firm believer in the dry mixing of concrete, but having seen several thousand cubic yards mixed dry and

placed to a depth of 6 ins. over a reservoir bottom, and afterward over 3 000 cu. yds. mixed wet, in the proportion 1: 3: 6, and placed in the other half of the same reservoir, he became a firm advocate of wet mixing. Mr. Christian.

A careful force account was kept on the last 3 000 cu. yds., in order to ascertain the actual cost per cubic yard. The wages paid were: Foremen, $2.50; laborers, $1.35; and teams, including driver, $4.00 per day. To the cost of each item in the force account 9% has been added for superintendent, timekeeper, office help, etc., and is included in the figures here given. The cost of blasting is not included, because the contractor was paid for excavating the rock, so that the cost begins with the loading and hauling to the crusher. The average haul was 400 ft. to and from the crusher.

The cost per cubic yard is subdivided as follows:

Sand	$0.40		
Natural cement	1.18		
			$1.58
Loading and hauling to crusher		$0.27	
At crusher		0.21	
Rent of "		0.01	
Coal at "		0.05	
Hauling from "		0.16	
			0.70
Concreting gang:			
Foremen		$0.05	
Laborers		0.54	
Teams		0.09	
			0.68
Total			$2.96

Alfred F. Harley, M. Am. Soc. C. E. (by letter).—The "theory of concrete" is certainly a subject upon which civil engineers should be well informed, but, nevertheless, the fact remains that the execution is invariably left to a contractor, who, while carrying out the instructions of the civil engineer, is required to guarantee the success and perfection of the work. Mr. Harley.

On one occasion the writer was consulted by the proprietor of a livery stable who had taken from a local contractor a sub-contract for hauling cement, sand and broken stone by the cubic yard, and, upon presenting the bill, was informed by the contractor that inasmuch as the concrete had required 1 part of cement, 3 parts of sand and 6 parts of broken stone, making a total of 10 parts, while the actual number of square yards of concrete laid, 6 ins. thick, was approximately 60 000 sq. yds., or about 10 000 cu. yds. of concrete, and consequently

Mr. Harley. the stable proprietor could only have hauled 1 000 cu. yds. of cement, 3 000 cu. yds. of sand and 6 000 cu. yds. of broken stone. All of which argument the livery stable proprietor followed very clearly in mind, but failed to check with the actual number of loads the teams had carried, even when taking into consideration the assumption that a quantity of the material might have been dropped along the road. The writer was not made a party to the settlement finally reached, but the shrinkage of the material when rammed was certainly not taken into consideration.

The writer cannot agree with Mr. Tillson as to the economic use of a machine-mixer for street concrete work. The figures Mr. Tillson gives should at least be reversed, 7 cents being, in the writer's estimation, a minimum figure for hand-made concrete and 10 cents a minimum figure for machine-mixed concrete. Neither is the result equally good, in the writer's opinion. The wheeling for any distance in a barrow tends to bring the mortar to the bottom, while the most valuable part of the concrete, commonly called "the slush," is wasted along the run-boards and could not be picked up and used before the initial set had taken place; while the amount of water cannot be regulated as nicely in the closed trough of the machine as when allowed to run on the concrete from a hose as it is turned over by hand.

Where continuous shoveling has to be performed, as with concrete mixed by hand, it is infinitely less fatiguing to the laborer if a steel plate is used to shovel on instead of boards. Gravel is also much more easily shoveled than broken stone.

Mr. Dunham's remarks in regard to the use of broken stone and the dust obtained in breaking are interesting. In gravel deposits there is found frequently a good quality of sand in such proportion as to fill entirely the voids between the gravel. A great many civil engineers require the material to be screened, and the sand and gravel used separately in the proportion specified. Lately, this class of material has been used by a Government official without screening, and with very good result; and it is only reasonable to suppose that at the time the gravel deposit was formed, the voids were filled entirely with the fine particles of sand washed into as close a position as they could very well occupy.

The advantage in hand-mixed concrete, in the writer's estimation, is not only in cost, but in the rapidity with which the concrete is put into position after being mixed, and the proximity of all the materials to their final resting place. One practice which the writer very firmly believes in following, is to throw a few shovels full of mortar, when mixed on the board, over the top of the last batch of concrete just previously laid, and by this means fill the interstices flush with the surface of the concrete. It might be claimed that a batch would then

contain less mortar than it should, but this is not so, inasmuch as the operation is repeated over each batch thrown into position. Mr. Harley.

In New Orleans, during the execution of some contracts, the writer had an opportunity to run a hand-concrete gang at the same time that another firm of contractors was operating a concrete-mixer gang, and not only in the employment of less men, but also in the number of square yards laid during the day, were the results of the two methods under discussion apparent and decidedly in favor of the hand-mixed concrete.

A day's work, in running three boards extending the full width of the street, would average about 900 sq. yds. of concrete 6 ins. thick, which result is all that could be wished, from the following gang:

6 wheel-barrow men—broken stone or gravel.
3 " " " —sand.
1 " " man—cement.
2 men opening cement.
7 " dry mixing.
8 " taking off boards.
3 " tamping concrete.
3 " grading concrete.
1 man attending to run planks.
3 water boys.
1 or 2 extra men, or, in all, say, 40 men.

The writer believes that Mr. Hall will find it quite customary in Europe to use broken stone, gravel, sand and cement in concrete, in certain quantities.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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PAPERS AND DISCUSSIONS.

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INSUFFICIENT PROVISION FOR COUNTER-STRESSES IN RAILROAD BRIDGES.

Discussion.*

By Messrs. HENRY B. SEAMAN and HENRY S. JACOBY.

Mr. Seaman. HENRY B. SEAMAN, M. Am. Soc. C. E.—The difficulty mentioned by the author arises from undue refinement in bridge design. The use of wheel concentrations implies that we are designing for some precise loading of the present time, while, as Mr. Prichard intimates, our designs should be an approximate provision for future conditions. The use of wheel concentrations practically submits the loading to the criticism of men who are not engineers, but who, nevertheless, exercise sufficient authority to restrict the loading. This condition would be avoided, to some extent, if a satisfactory equivalent load could be adopted.

The method suggested, of doubling the load, will avoid the difficulty described, though it seems to be rather an extreme measure. Fifteen years ago bridges were designed for 25 000-lb. concentrations, and to-day the actual concentrations in some instances reach nearly 50 000 lbs.; but this does not mean that they will reach 100 000 lbs. in the future. The larger part of the present increase took place between 1885 and 1890, and the increase since the latter date has been comparatively slight, though its adoption has been more general. The

* This discussion (of the paper by Henry S. Prichard, M. Am. Soc. C. E., printed in *Proceedings* for August, 1899), is printed in *Proceedings* in order that the views expressed may be brought before all Members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to November 24th, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

weights of locomotives having been increased about as much as the present gauge and car clearance will permit, any future increase will arise from a change of the distribution rather than from any material increase of concentration, and there seems to be no possibility of doubling the loads for which the heaviest bridges are now designed. Mr. Seaman.

Henry S. Jacoby, Assoc. Am. Soc. C. E. (by letter).—The subject of counter-bracing does not usually receive the attention it deserves in the design of bridge trusses. Sometimes the only rule followed is to put a counter in a panel on each side of those in which theory requires their insertion for the specified dead and live loads, without regard to the effect of impact or the future increase of loads. In some cases this may be sufficient, but in other cases the provision is wholly inadequate. This rule is certainly not a rational one. Mr. Jacoby.

The specification recommended in the paper is in accordance with the best practice, known to the writer, for the design of counters. The increase of 100% in the live-load stresses, besides the addition for impact, is not simply to provide for the gradual increase of the live load until it may reach that figure before the life of the bridge expires, but to provide for occasional loads of unusual magnitude, so as to avoid the necessity of shoring up bridges for such requirements, except those which may be nearing the end of their service.

As pointed out, the need for a better provision for counterstresses is greater in trusses with curved chords than for those with parallel horizontal chords. It seems that some account should also be taken of the stresses due to the overturning moment of the wind, for a reversal of stress due to insufficient counter-bracing caused by the wind in conjunction with the live load may be just as serious in its results as if it were caused entirely by the live load.

Under the usual specifications, the stress due to the wind may be neglected in the design of any member unless its magnitude exceeds a given percentage of the dead plus live-load stresses, its character being the same. When, however, the character of the wind and live-load stresses is different from that of the dead-load stress it may require but a relatively small amount of the wind stress to cause a reversal of stress, and this fact often escapes notice on account of the too prevalent custom of failing to compute any minimum stresses.

Again, the addition of a counter-brace in a panel may, in turn, not only modify the magnitude of the minimum stress in an adjacent vertical, but may actually change the character of that stress, and thereby affect its design to a greater or less extent.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

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IMPACT TESTS OF STRUCTURAL STEEL.

Discussion.*

By C. M. BROOMALL, Jun. Am. Soc. C. E.

Mr. Broomall. C. M. BROOMALL, Jun. Am. Soc. C. E. (by letter).—All impact testing machines necessarily give truly comparative results only under similar conditions of striking hammer, supports, foundations, etc. These conditions are not realized in practice, and the best that can be done is to so arrange matters that the greater portion of the energy of the blow will be absorbed by the specimen. If the proportion of energy absorbed by the specimen is very large, the error caused by the difference in the conditions of the apparatus will not affect materially the accuracy of the results. In the machine designed by the author it is believed that the energy absorbed by the specimen is more than 90% of the whole, so that the results given by it must be quite accurate.

It seems to the writer, however, that a machine might be so designed that the specimen would absorb practically all the energy of the blow. Whether or not the mechanical details would be too complicated is another question. The suggestion is, to make use of a differential method of measurement, and, instead of mounting the specimen upon rigid supports, to mount it upon another pendulum

* This discussion (of the paper by S. Bent Russell, M. Am. Soc. C. E., printed in the *Proceedings* for August, 1899) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to November 24th, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

initially at rest. If, then, after the impact, the rise of both pendulums be measured, the data are obtained from which to calculate the energy absorbed. This energy must have been entirely spent upon the specimen and its clamps. As these clamps are attached to the second pendulum, their weight must be added to it. The only energy not spent upon the specimen will be that absorbed in deflecting and producing angular motion of the clamps. Mr. Broomall.

This differential method virtually amounts to suspending Mr. Russell's whole machine as a pendulum from the same axis as the striking pendulum, and so adjusting matters that the centers of percussion of the two pendulums are coincident with the point of impact.

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THE ARTIFICIAL PRESERVATION OF RAILROAD TIES BY THE USE OF ZINC CHLORIDE.

Discussion.*

By Messrs. N. W. EAYRS, O. E. SELBY and O. CHANUTE.

Mr. Eayrs. N. W. EAYRS, M. Am. Soc. C. E. (by letter).—The author has presented a deal of valuable information on the subject of preservation of railroad ties, but it is to be regretted that the paper was not extended to include ties and timber used in bridges and viaducts. The cost of renewing ties on such structures and under traffic is very much greater than that of tie renewals on ballast, and every year added to the life of such ties is more valuable than in the case of common ties.

The writer's experience in recent years has been chiefly in the maintenance of bridges, viaducts and elevated structures, and he has been surprised to find that so little effort has been made to protect bridge ties and floors from the action of the weather. For a number of years past it has been the writer's practice to paint bridge ties, and the result has been so satisfactory that he is inclined to doubt the advisability of treating such ties by any other preservative process. One great objection to using the ordinary processes for preserving such timber is, that, as universally admitted, timber should not be cut after treatment; it was suggested to the writer by the management of one preserving works that the ties should even be bored for spikes before treatment. The expense of such rehandling, and the extra freight charges, would be prohibitive.

* Continued from September, 1899, *Proceedings*. See April, 1899, *Proceedings* for Paper by W. W. Curtis, M. Am. Soc. C. E., on the subject.

Mr. Eayrs.

The writer's experience on the St. Louis Bridge is an instance of the value of painting ties. The railway floor of this bridge was changed to a tie floor in 1888; the ties used were 7 x 7-in., long-leaf yellow pine, carefully selected. All daps were painted before the ties were placed, and also all surfaces where wood came in contact with wood. After the floor was finished the ties were given two coats of red oxide of iron and linseed oil. These ties are in perfectly sound condition to-day. Unfortunately, tie-plates were not used when the ties were first laid, and it is now necessary to renew them on account of "spike killing" and wear; otherwise the ties would be good for 5 years longer, if not more.

In 1897, the writer renewed the entire floor of the elevated portion of the St. Louis Merchants Bridge Terminal Railway. This floor was laid in 1891 with oak ties, unpainted and untreated. Six years afterward more than 50% of the ties were so badly decayed that nothing was left but a shell. The traffic over this structure had been very light up to 1896, and, consequently, the ties had suffered no mechanical injury. The timber used in relaying was long-leaf yellow pine, and was painted with one coat of mineral paint after framing, and one coat after being placed in the track. The cost of painting the ties, two coats, averaged $11\frac{3}{4}$ cents per tie. Tie-plates were used on this structure, and, judging from the experience on the St. Louis Bridge, these ties ought to last 15 years. About 75 ties were treated by dipping into a bath of hot Woodiline, and about 150 others were painted with Woodiline applied hot. The dipped ties were allowed to remain in the bath ten minutes; the amount absorbed was $22\frac{1}{2}$ galls. per 1 000 ft. B. M., or about $1\frac{1}{3}$ galls. per tie. The cost of the work was 52 cents per tie for material, and about $1\frac{3}{4}$ cents per tie for labor in handling, exclusive of the cost of the tank, steam connection and fuel.

Taking into consideration the cost of the process, the uncertainty of getting the proper amount of preservative in the timber, and the necessity of treating after the framing has been done, it is the writer's opinion that for bridge ties there is very little more economy in any of the preserving processes now in use than in painting.

Mr. Selby.

O. E. Selby, Jun. Am. Soc. C. E. (by letter).—Table No. 15 and the diagram, Fig. 3, were prepared by the writer during an investigation of the question of timber preservation for the railroad system with which he is connected, for the purpose of arriving at a rational and ready means of comparing the economy of different methods of preservation.

The diagram, omitting the four-sided figures lettered *A*, *B*, *C*, etc., is plotted from Table No. 1, page 195, of Bulletin No. 9, United States Department of Agriculture, Division of Forestry. The abscissas represent the cost in cents of each renewal, and the diagonal lines represent the intervals between renewals. The ordinates show the annual

TABLE No. 15.

	Kind of Tie and Treatment.	Cost of.			Total cost in track.	Probable life. Years.	Annual charge. Cents.	Life Necessary to Equal White Oak Tie.	
		Tie delivered.	Treatment.	Putting in track.				At 38c.	At 50c.
A....	First quality white oak, untreated. Present prices...	35 to 38	.....	12 to 15	47 to 53	8 to 10	6.1 to 8.2	.....	.....
B....	" " " " " Future " ...	50 " 55	.....	"	62 " 70	8 " 10	8.0 " 10.8	.....	.....
C....	Inferior oaks, untreated.....	30 " 34	.....	"	42 " 49	4 " 5	9.7 " 13.8	.....	.....
D....	" " Burnettized.....	30 " 34	16 to 20	"	58 " 69	10 " 14	5.9 " 8.9	$11\frac{1}{2}$	9
E....	Hemlock, gum, etc., untreated.....	28 " 32	.....	"	40 " 47	3 " 4	11.3 " 16.7	.....	.....
F....	" " " Burnettized.....	28 " 32	16 to 20	"	56 " 67	9 " 12	6.3 " 9.4	11	$8\frac{1}{2}$
G....	" " " " and tie-plates.....	28 " 32	35 " 40	"	76 " 87	11 " 14	7.7 " 10.5	$16\frac{1}{2}$	12
H....	Beech, untreated.....	30 " 32	.....	"	42 " 47	3 " 4	11.8 " 16.7	.....	.....
J....	" Burnettized.....	30 " 32	16 to 20	"	58 " 67	12 " 15	5.6 " 7.5	$11\frac{1}{2}$	9
K....	" minimum life, Burnettized.....	30 " 32	16 " 20	"	58 " 67	9 " 10	7.5 " 9.4	.....	.....
L....	" " and tie-plates.....	30 " 32	36 " 40	"	78 " 87	14 " 16	7.2 " 8.8	17	12
M....	Chestnut, untreated.....	30 " 34	.....	"	42 " 49	5 " 6	8.3 " 11.3	.....	.....
N....	" " and tie-plates.....	30 " 34	20	"	62 " 69	12 " 14	6.3 " 7.7	12	9
O....	Cedar, "	35	.....	"	47 " 50	5 " 6	9.3 " 11.5	.....	.....
P....	" " and tie-plates.....	"	20	"	67 " 70	12 " 14	6.8 " 7.8	13	10
Q....	Inferior yellow pine, untreated.....	45	.....	"	57 " 60	4 " 6	11.1 " 16.9	.....	.....
R....	" " " creosoted.....	"	45	"	102 " 105	20 " 22	7.7 " 8.4	23	$17\frac{1}{2}$
S....	" " " vulcanized.....	"	23 to 30	"	80 " 90	15 " 16	7.4 " 8.7	16 to 18	12 to 13
T....	" " " " and tie-plates.....	"	43	"	100 " 103	16 " 18	8.5 " 9.5	24	17

Mr. Selby.

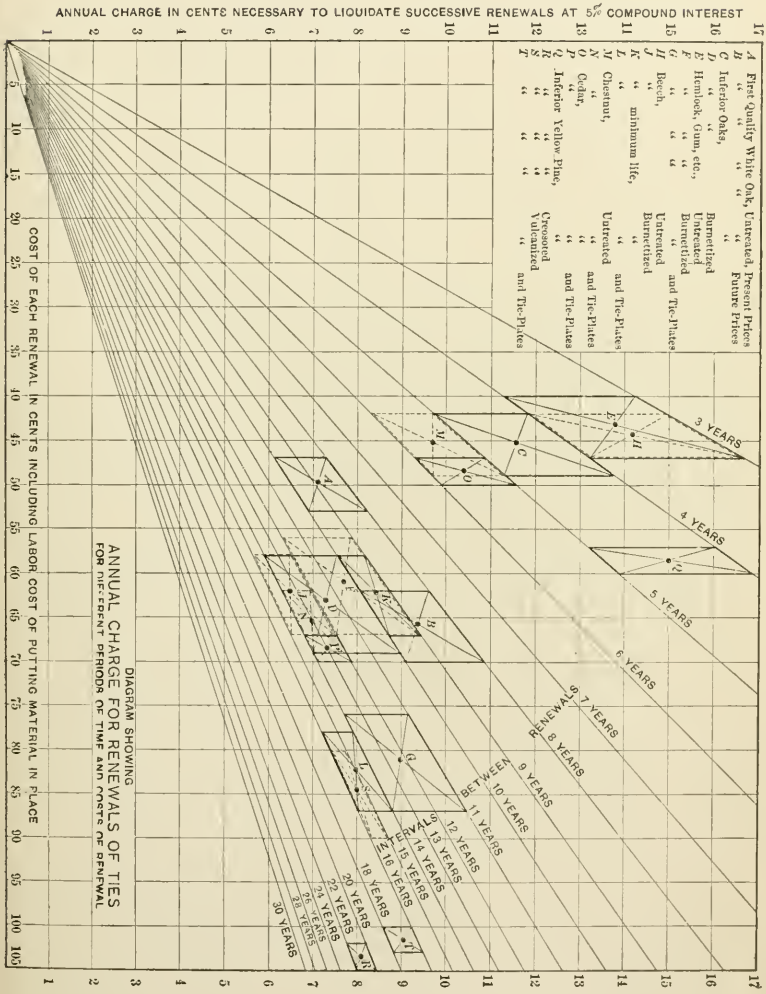

Fig. 3.

Mr. Selby. charge in cents, based on 5%, compound interest, necessary to liquidate the succeeding renewals at the intervals shown. In other words, it is the amount which it would be necessary to set aside each year and invest at 5%, compound interest, in order to equal, in the end, the cost of all renewals reckoned at the same rate of interest.

The diagram shows graphically and exactly the comparative ultimate economy of various methods involving various lengths of life and initial costs. The only approximation involved is in the assumed lengths of life and assumed costs. Comparisons can be made for structural timber by taking the costs and annual charges in dollars per 1 000 ft. B. M., instead of in cents per tie.

While it is known that most railroads can now obtain money at less than 5%, it is believed that the addition of general expenses, taxes, costs of financing, uncertainties in distribution of expenses, etc., makes a 5% basis of comparison a proper one, especially as it works to the disadvantage of the longer-lived ties and processes, as to the durability of which there is necessarily much uncertainty.

The table was prepared to include the principal kinds of ties and methods of treatment available on a large scale on the system in question. There are several species of wood, such as catalpa, sassafras, black walnut, etc., which are known to be very durable and valuable for cross-ties, but which are not available in large enough quantities to have much effect on the tie supply of a large system, and they were not included in the comparison.

The basis of comparison is the first-class tie, of white oak, burr oak or post oak, filling the present specifications of the system, costing 35 cents to 38 cents each, and lasting in the track from 8 to 10 years, under reasonably favorable conditions. In view of the increasing scarcity of such first-quality oak, and rapidly advancing prices, a comparison is also made on a basis of the probable early future cost of 50 cents each. The assumptions in the table, as to cost of ties, treatment, putting in track and probable life, are based on reports from division engineers, and information from other sources, including Mr. Curtis' paper, applicable to the middle western territory covered by the system. Limits were used rather than averages so as to show the possible range covered by each process.

The column headed "Total Cost in Track" is made up by adding together the cost of the tie delivered, the cost of treatment, if any, and the labor cost involved in removing the old tie and putting in the new one. It is important that this item of cost of putting in track be included in all comparisons, as it is certainly an expense involved in each renewal, and its omission gives an unfavorable advantage to the short-lived ties and processes. The column headed "Annual Charge" is taken from the diagram for the preceding data of total cost in track and probable life. The last two columns of the table show the length

of life which must be realized by the different processes at the assumed costs in order to equal in economy a white oak tie, first, at present prices; second, at future prices. This was obtained from the diagram by following out the horizontal line representing the average annual charge for the oak tie to its intersection with the vertical line representing the average cost of the treated tie. The year lying nearest this intersection is the required life. Mr. Selby.

The use of metal tie-plates, with treated and untreated ties, was considered, together with other methods of preservation, and the results show it to be a most promising method with soft wood ties. The cost of tie-plates was assumed at 10 cents each, including the cost of their transportation to the site and putting them in. This may seem high for small plates, but it is likely to be realized with the advancing prices of steel.

Two assumptions were made for Burnettized beech ties, the first based on their claimed and probable life of 12 to 15 years, and the second on a life of 9 to 10 years, which seems a certain minimum, in view of experience with the process and of improvements now being tried. The range of cost of vulcanizing was made large to cover both the present cost and the future reduced cost of the process if it proves successful and works are established in the West. In view of developments in the tie market, since the table was made, the price of cedar ties was probably taken too low.

The data in the table have been plotted on the diagram, each kind of tie being represented by a four-sided figure, within the limits of which will be found the actual annual charge, the intersection of the diagonals locating the average. Those plotting nearest to the base line show the lowest annual charge, and hence the greatest economy.

The one which plots lowest is the beech Burnettized, but if its life is proven to be only 9 or 10 years it plots much higher, but is still lower than the white oak tie at 50 cents. The chestnut tie with plates, the cedar tie with plates, and the inferior oaks Burnettized, all show greater economy than the white oak tie at 38 cents delivered. All the processes of treatment plotted show greater ultimate economy than a white oak tie which costs 50 cents delivered and lasts only 8 to 10 years, indicating that the time is rapidly approaching when some form of preservation will be imperative.

On the face of the diagram the best showing is made by the Burnettizing process, and by untreated, long-lived, soft wood ties with plates.

The flat inclination of the lines representing the longer periods of years shows that the amount we are justified in spending for a tie increases rapidly with the life realized; that is, the annual charge increases more slowly in comparison with the first cost for the long-lived ties than it does for the short-lived ties.

Mr. Selby. The cost and probable life in any individual case may easily fall outside the limits plotted, but in any case it is easy to follow out the horizontal line corresponding to the cost in track and probable life of ties in use to find the life which it will be necessary to realize from a process of given cost, or the amount which it is justifiable to pay out to secure a tie of a given life.

The writer's investigation resulted in a recommendation for the purchase of a considerable number of inferior oak ties and their treatment by Burnettizing, and the purchase of a like number of cedar and chestnut ties and their use with tie-plates; the number recommended being large enough to be considered experimental on a large scale.

There is much promise in a modified Burnettizing process, now being investigated by a leading tie-preserving company, consisting of the injection of both creosote and zinc chloride. This is likely to prove durable, but may be too expensive. The writer agrees with others who have taken part in the discussion that much of the hesitancy of the managements in authorizing the use of treated ties is on account of the uncertainty which exists as to the actual life, under varying conditions, both of ties now in use and of the treated ties.

Certainty, in this direction, is only to be attained by careful records of all ties in the track, showing length of service and actual cause of removal. Such records would be of value in determining, not only the necessity for and the best processes of treatment, but in indicating the value of improvements in any given process.

Mr. Chanute. O. Chanute, Past-President, Am. Soc. C. E. (by letter).—This paper chances to be most timely, for its publication concurs with one of those swift advances in prices which take place when it is suddenly discovered that the supply has been overtaken by the demand. One year ago ties were selling about as cheaply as they had been for 10 years past. In Chicago, for instance, white oak ties were quoted at 45 cents and cedar ties at 36 cents, while now the price of white oak is 60 to 65 cents, and of cedar 50 cents, with talk of further advance. Railroad men, therefore, are considering now the use and treatment of the cheaper and more perishable kinds of wood, which are still abundant.

There is no longer any question in the mind of the writer that chemical treatment, applied thoroughly and skilfully, will more than double the life of the inferior woods, and make it equal or superior to that of the more durable woods laid in their natural state. All the European experience has proved this, and in spite of some failures, due to ignorance of all the conditions of success, American experience is seen to confirm that of the Europeans.

The writer drifted into this business by chance some 14 years ago. He sees now that he made some mistakes in the beginning, and had much to learn concerning the idiosyncracies of various woods, so that his experience may be of value to those who may carry on such

work hereafter. His first mistake consisted in the apprehension that strong solutions would make the ties unduly brittle, and in believing that solutions of zinc chloride of 2° Beaumé, or less, would penetrate farther into the wood than stronger and less limpid solutions. Mr. Chanute.

In 1879, Mr. B. F. Bischoff, then in charge of the tie-preserving works of the "Empress Elizabeth Railway," of Austria, read an elaborate paper before the Austrian Railroad Club, in which, after describing the then state of the art, he gave an account of some experiments which he had made, and which went to show that zinc-chloride solutions of 1.7° Beaumé could be forced into the wood further, and that they protected the ties better, than solutions of 3° Beaumé. He instanced, as proof, the results cited by Mr. Funk, in a paper which has remained a classic, concerning the life of ties treated with solutions of 2° Beaumé, and he stated that the "Empress Elizabeth Railway" had changed over from the use of zinc-chloride solutions of 3° Beaumé to those of 1.7° Beaumé. This practice, which seemed to promise some economy, was followed by many German railways, but the results were not found satisfactory. It was ascertained that these railways did not do as good work as others, which used stronger solutions, and the German specifications of 1898 and 1899 require the use of solutions of 3.5° Beaumé, and the injection of 0.50 to 0.55 lbs. of dry zinc chloride per cubic foot of wood.

In consequence of the fact that American ties get less seasoning than is usual in Germany, the writer began treating ties at Chicago in 1886 with solutions of 3.5°Beaumé, at 80° Fahr., and he has gradually increased the strength to 5° Beaumé, having in the meantime learned by experiment how to inject these strong solutions without making the tie brittle. Not one of them has broken in the track. He is now injecting two and a half times as much chloride of zinc as he did at the beginning, and has every reason to believe that the results, with ties treated since 1896, will be entirely satisfactory.

His second mistake consisted in assuming that there was no great difference in the quantity of solution absorbed by each individual tie in a run, and that they might be treated with success in the order of their arrival. Unfortunately, ties arrive in Chicago by vessels in varying condition; some have been cut longer and are better seasoned than others, while particular cargoes, or parts of cargoes, have been rafted out to the vessels, and are saturated with water. It was believed at first that these varying conditions could be overcome simply by changing the strength of the solution, from day to day, to correspond, and that satisfying results would follow. This proved to be an error, and now refractory ties are unloaded upon the ground to season; some 70 000 having been so treated in 1898, and some 120 000 in 1899. Moreover, hundreds of experiments with individual ties, made by weighing them just before and after treatment, have established the

Mr. Chanute. fact that there are surprising differences in the receptivity of various kinds of wood, and of various ties cut at the same time and from the same forest. Some will absorb three times as much as others, and, in extreme cases, when both seasoned and unseasoned ties are tested, the differences are as 8 to 1. The conclusion drawn was, that, in order to inject the minimum of zinc chloride guaranteed in every tie, it was necessary to inject on an average about three or four times this quantity, and this is now being done.

This difference between individual ties was found to be greatest with tamarack, probably in consequence of the varying amount of resin contained therein, and, after many experiments, the treatment of ties of this class was reluctantly abandoned.

Thus it is seen, that, simple as the process appears, theoretical knowledge alone is not sufficient to insure doing good work. Practical experience is required constantly, and this is acquired best with a small experimental plant, with which various methods may be tested without interfering with the regular runs.

It is unfortunate that various changes in the personnel of the maintenance-of-way departments of some of the railroads first adopting tie-preserving methods in this country should have produced discontinuity in the records, but these are sufficient, as Mr. Curtis has shown, to prove that inferior woods are made more durable than white oak, and we may expect still better results now that experience has taught us how to do the work better.

The writer is satisfied that his present practice of injecting from 0.50 to 0.55 lbs. of zinc chloride per cubic foot of wood is an improvement over his earlier methods. Also, that the injection of three separate solutions instead of two is a decided improvement. He believes the zinc-tannin process, thus applied, to be better than the zinc process alone, and that experience in regions of abundant rainfall will prove this to be a fact. Burnettizing, as we know, has been abandoned in England, while it is a success in Germany, probably in consequence of the difference in rainfall in those two countries.

In the hope of learning still more about the anomalies of various woods, and the best methods of treating them, the writer is now about to visit Europe, where far more experience has been accumulated than in America, and he regrets that the present publication of this valuable paper will prevent this discussion from containing the results of his pending tour of observation.

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GENERAL CRITERION FOR POSITION OF LOADS CAUSING MAXIMUM STRESS IN ANY MEMBER OF A BRIDGE TRUSS.

Discussion.*

By L. M. Hoskins, Esq.

Mr. Hoskins.

L. M. Hoskins, Esq. (by letter).—It was no part of the object of this paper to advocate the use of wheel concentrations as a basis of bridge design. This kind of specification has been in general use for a considerable number of years, and, doubtless, will continue to be used for a long time to come, in spite of the efforts of its opponents to supersede it. Even if some different specification should be adopted generally, many engineers of railroad companies probably will think it advisable to apply the wheel-concentration method in estimating the effect of new and heavier train-loads on existing bridges. In view of these facts it seemed worth while to present what appeared to the author to be a material simplification in the treatment of the general problem of computing stresses due to any series of loads whatever.

The members who participated in the discussion at the last Annual Convention of the Society were unanimous in condemning the method of wheel-concentrations. They were, however, by no means agreed as to what method should replace it. Thus, the method advocated by Mr. Morison was condemned by Mr. Waddell as complicating matters worse than ever. And, while the majority appeared to favor

* Under the rules for publication, additional discussion on this paper, received prior to November 24th, 1899, will be published with the paper in the next volume of *Transactions*.

Mr. Hoskins.

some form of "equivalent uniform-load," each had his own opinion as to how this should be estimated. That this discussion does not represent the unanimous opinion of those engineers most directly interested in the question is indicated by recent correspondence in *Engineering News*.*

No one supposes that the method of wheel-concentrations can represent with exactness the actual distribution of loads upon a bridge. Its advocates merely contend that it comes a step nearer than other methods to representing actual conditions. The truth of this contention is tacitly admitted by most of those who condemn the method; the correctness of every proposed method is tested by a comparison of its results with those of the wheel-load method.

The only real objection to the wheel-concentration method, as compared with other proposed methods, is the difficulty of the computations involved. How serious this objection is can be judged only by one who is thoroughly familiar with the best methods of computation. Wheel concentrations appear to present no serious difficulties to the computer who familiarizes himself with the best methods of dealing with them.

Aside from whatever practical value it has, the problem discussed in the paper possesses considerable mathematical interest. The properties of influence lines given by Mr. Cilley and Mr. Kunz are interesting contributions to the discussion. With reference to the comment of Mr. Kunz, on the general rule for drawing influence diagrams, it may be remarked that when the moment-center falls beyond convenient limits, the ordinates at C and D can be computed readily by the general values given on page 78.

* July 13th and September 7th, 1899.

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PILE-DRIVING FORMULAS: THEIR CONSTRUCTION AND FACTORS OF SAFETY.

Discussion.*

By Charles H. Haswell, Esq., M. Am. Soc. C. E.

Mr. Haswell.

Charles H. Haswell, M. Am. Soc. C. E. (by letter).—In response to the remarks of Mr. Crowell, on the results and deductions of the author's experiments to ascertain the relative impact of falling weights from different heights, it was not essayed, nor was it held to be at all practicable, to determine exact measures of impact with an instrument possessing such retarding elements as a spring-balance, supplemented with a detent. It was essayed only to ascertain the relation and ratio of the impact of the weights from different heights of fall, and as like causes produce like effects, the relation of the results, satisfactorily determined by repeated operations, were just as conclusive as if obtained with the exactness of the requirements of normal results.

Regarding the final penetration of 0.5 in. being assumed as the base of the formula submitted, it was taken, inasmuch as it is not only the ordinary limit of practice, but it is a sufficient refusal and the practically proper one; as the difference between it and a less depression is not only difficult to observe, but the attainment of it is of little relative value with the usual factor of safety, and when effected, it results frequently in the splitting of the piles; further, if it is different, the factor can be altered to meet the conditions, and in any for-

* Under the rules for publication, additional discussion on this paper, received prior to November 24th, 1899, will be published with the paper in the next volume of *Transactions*.

Mr. Haswell. mula, if the final penetration is not a fixed element, computation of the capacity of a pile is emasculated, and the judgment of the driver from visual observation is the only result whereby to assign its capacity.

A mean factor of 4.5 is assumed, and when it is considered that the resistance of a pile in one hour after being driven, is, by the subsidence of the soil around it, very much increased, it is held to be a full measure of safety. In moist sand, with a rough pile of oak or pine (not spruce), a factor for a quiescent load is, in the author's opinion, practically unnecessary, and this opinion is based upon exceptional opportunities to observe the effect of such conditions.

It is not the author's purpose to discuss the application of *vis viva* in this case, it is preferred to give observed results. The resistance an instrument opposes to a falling weight is assumed as a measure of its force, whether it is a spring-balance or a supporting pile.

In further support of his remarks, Mr. Crowell submits a diagram illustrative "of the characteristic plotting of the respective formulas" of Wellington and the one submitted, and that "it will be seen that for a fall of only 2 ft., the indicated safe load is one-third as great as that indicated for a fall of 18 ft., nine times as great." Now, as the formula gives but three times for the 2 ft., and as $\sqrt{2\,g\,18}$, is just three times that for 18 ft., the formula agrees with the illustration, and the result is not nine times, as asserted.

Mr. Gould is inclined to believe that the use of the $\sqrt{H}$ is correct. His opinion, from his capacity and great experience, is entitled to much consideration.

Mr. Howe recites that by the published experiments of Mr. McAlpine, the sustaining power of a pile is increased in the ratio of the square root of the fall of the ram. This also is in support of the formula used, and is wholly at variance with the formula of Mr. Wellington.

Mr. Le Conte's deduction and experience, regarding the second driving of a pile after an interval of a day in sandy soil, is at variance with the author's experience, and as to the value of the Dutch formula, the author fails to recognize the utility of considering the number of blows of a ram, when the effect of the final blow or impact of it is the measure of the resistance of the pile. The tenacity of a nail in wood is greater when driven at one blow than by a number of them, for with one blow there is but one resilience of the disturbed fibers of the wood, which resistance is the element of the tenacity of the nail, and a repetition of blows is a repetition of the resilience and consequently a weakening of it.

"Molesworth on Pile-Driving" gives a table of the force of a blow of a falling weight $= V\,W$.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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THE GROINED ARCH AS A COVERING FOR RESERVOIRS AND SAND FILTERS: ITS STRENGTH AND VOLUME.

Discussion.*

By Messrs. ALLEN HAZEN and WILLIAM B. FULLER.

ALLEN HAZEN, Assoc. M. Am. Soc. C. E.—In looking over some plans and photographs of vaultings recently, the speaker was surprised to find how seldom the groined arch has been used, considering its marked advantages in economy and strength. Mr. Hazen.

In the Roman reservoirs of Constantinople† an arch resembling the groined arch was used freely, but it was not a true groined arch. It was made by springing narrow arches between the piers in each direction and dividing the area up into a series of squares which were covered by flat domes. This form of construction is often seen in architecture to-day. The ceiling of the reading room in the Public Library in Boston is a very handsome example of it.

An important modern application of this form of vaulting is at Warsaw, Russia, where filters and reservoirs of ten or twelve acres are covered in this way.

Many covered reservoirs and filters have been constructed with cylindrical arches resting upon narrow arches sprung between the piers in one direction only. This form of construction is common in England. The earliest Continental reservoirs and filters were also

* Continued from September, 1899, *Proceedings*. See May, 1898, *Proceedings* for Paper, by Leonard Metcalf, Assoc. M. Am. Soc. C. E., on the subject.

† Forchheimer, "Wasserbehalter von Constantinopel," Wien, 1893.

Mr. Hazen. covered in this way. The older water-works vaultings at Berlin are of this class, and also the filters at St. Petersburg.

The more recent vaultings at Berlin, and elsewhere upon the Continent, are usually groined arches. The corners are often re-enforced with ribs, as Mr. Metcalf has suggested, and the speaker thinks there is a theoretical reason for this in the case of brick vaulting, although whether or not the additional strength is worth the trouble is another question. In the earliest covered reservoirs constructed in the United States, English precedent was followed, and cylindrical vaulting was used. The speaker thinks that the first large masonry-covered reservoir in America was at Newton, Mass. This reservoir was designed by the late Albert F. Noyes, M. Am. Soc. C. E., and Alphonse Fteley, Past-President, Am. Soc. C. E., acted as consulting engineer.

To William Wheeler, M. Am. Soc. C. E., belongs the credit of the introduction in America of the groined arch for water-works purposes, as has been stated by Mr. Metcalf. The speaker has had occasion to use it extensively in the new filters for the water-works of Albany, N. Y., and has had occasion to make computations similar to those which Mr. Metcalf describes. The volumes of groined arches have been computed in the speaker's office by William B. Fuller, M. Am. Soc. C. E., and others, by methods similar to those which Mr. Metcalf has used. These computations have been checked by graphical methods. The graphical computation of the volumes is not difficult, and the results check perfectly those obtained by the formulas.

The speaker is sorry that the paper does not give more in reference to the strength of the vaulting; he thinks that all the discussion of the subject relates to brick arches. In the case of concrete arches he is inclined to think that the strains involved are somewhat different from those in brick, and that the computations relating to brick arches will have to be taken with some allowance when applied to concrete.

This, perhaps, will depend upon the way in which the concrete is placed. Concrete is necessarily placed in lots, leaving joints at certain places, and the tensile strength of these joints is very little, practically nothing. The speaker has used joints following the summits, that is, dividing the vaulting up into a series of squares, each having a pier as a center. Under these conditions, and with the dimensions used at Albany, it is a question whether there is any arch action whatever in the vaulting. Changes in temperature expand and contract the concrete. With brick arches this expansion and contraction would probably result in a slight rise and fall of the crowns, corresponding to the differences in length of the material in the arches. With the concrete, the action is apparently different. The blocks are so rigid that there is no appreciable rise and fall of the crowns. With contraction, the joints will open slightly in some cases, making

it perfectly obvious that there is no pressure across them, and that the concrete blocks act as cantilevers; while with expansion, which occurs with concrete put in in cold weather and afterward warmed by summer heat, the whole mass of concrete has to push out at the sides. In some cases the cylindrical arch next to the outside wall may act as an expansion joint for the groined vaulting in the interior. Mr. Hazen.

The speaker has mentioned the cantilever principle simply as a proposition. He is not prepared to say that he thinks all vaulting acts in that way. He is not fully informed as to the method of construction used in the Wellesley vaulting, but his impression is that the concrete was not placed in the manner used at Albany, and that the joints were made at other places; and if that was the case it could not be expected to act in the same manner. On the Albany work the concrete was mixed by machinery, and the amount of concrete required to fill one of the squares was placed in a very few minutes, and made a solid block, without a joint of any kind.

Regarding the practical strength of the concrete vaulting, there can be no question. At first the speaker made certain rules in regard to teams driving over it, etc., but found it very hard to keep the contractors in order in that respect. After a little, teams were allowed to go over it freely, and also trucks, and later, rollers weighing three tons were taken over it repeatedly, and sometimes left on it, and all without any injurious results.

The cost of the Albany vaulting per square foot of area was as follows:

Concrete 0.78 cu. ft. at $6.30 per cubic yard, including centers	18.20	cents.
Piers	5.44	"
Earth filling and seeding	1.42	"
Manholes, entrances, fasteners, etc.	2.65	"
Total	27.71	"

The total cost, including piers, was less than 28 cents per square foot Various forms of steel and concrete construction were considered, before deciding upon the concrete vaulting, but the simple masonry was selected.

William B. Fuller, M. Am. Soc. C. E. (by letter).—It is well known that there are many advantages to be derived from a covered reservoir or filter-bed as compared with one open to the wind, rain and light, and the fact that a covering of such a nature is seldom used would indicate a false impression, even on the part of many engineers, that it is necessarily very expensive and should be classed among the luxuries, except in cases where it is essential to prevent the formation of ice or algæ. Mr. Fuller.

Mr Fuller. This is due perhaps to the fact that engineers, in designing masonry coverings, have been hampered by precedent. They look back to structures, some of which, as the author points out, date from the time of the Roman supremacy, and proceed to construct vaulting as laid down on lines developed 2 000 or more years ago. This is particularly true of recent vaulting in Europe, outside of France, and the engineer, even if he should desire to branch out on a more rational design, is confronted with these examples of vaulting, used merely for the protection of water from the action of the elements, but actually strong enough to carry railroad freight traffic.

The modern development of Portland cement concrete furnishes the engineer of to-day with a structural material easy to handle, particularly adapted to the construction of vaulted masonry, and differing widely in some of its properties from materials heretofore commonly used in such construction; and the writer wishes to emphasize a few points concerning the stresses which result in monolithic work of this nature.

The author translates and endorses a theory on the stability of groined arches dependent entirely on the non-existence of other forces than compression, and says:

"It is apparent, of course, that this method makes no allowance for the greater stability of the structure due to the monolithic character of the masonry when constructed of concrete; which stability is no doubt materially increased thereby."

The writer has repeatedly noticed this tendency with engineers in designing structures of concrete; they are made similar to structures in brick or stone, and the added stability due to other properties than those possessed by such materials is dismissed with the thought that the structure is all the safer. We would not think of substituting steel for cast iron and taking into account only the properties of the latter.

Monolithic concrete is an independent building material, and surely, in such a position as in groined arch masonry, is entitled to separate consideration; the stresses in such a structure must be very complex, but possibly their nature is somewhat as follows:

Consider a groined arch unit as shown by the author in Fig. 4, and cover its upper surface with a uniformly distributed load, the result must be a tendency to lower the outside edges, and, in a single unit, such as we are now considering, it can be seen that this tendency must be resisted by the tensile, cross-breaking or shear stresses in the mass of the unit itself. This action will be understood more readily if we think of the unit as made up of a stack of square, thin plates of varying areas piled symmetrically on top of the pier, as in Fig. 15. If we go a step further, and remove the center of these plates, we have

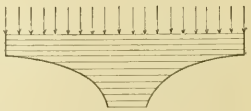

Fig. 15.

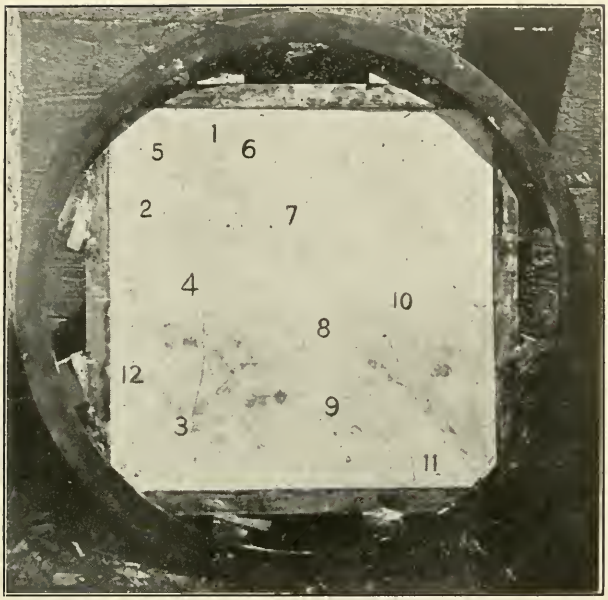

FIG. 1.

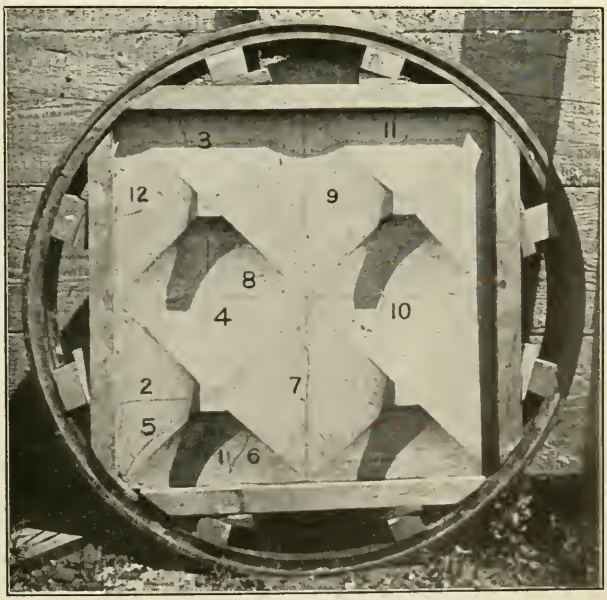

FIG. 2.

the result indicated in Fig. 16, in which most of the cross-breaking stresses are replaced by tension in the rings, and we have a case somewhat analogous to that of a dome. Mr. Fuller.

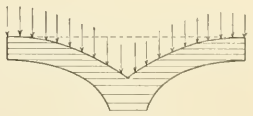

Fig. 16.

The placing of these units together to form a groined arch vaulting also brings into play compressive stresses, but as there must be a perceptible lowering of the keystone before the arch action takes effect, it is seen that, if the groined arch unit is capable of resisting the tensile stresses, the arch action may not be brought into play at all and is certainly only a secondary consideration. In such a case the stability of the structure is added to in just such proportion as these tensile and other stresses are brought into play.

These illustrations are enough to induce a disbelief in the applicability of the author's theory to monolithic concrete vaulting, and the writer believes such a theory is equally inapplicable to brick or stone construction because the strain represented above as being resisted by the mass of the unit section, in this other case would be resisted by the outside abutments of the structure.

It is to be regretted that in demolishing a theory on which formulas are based, other formulas cannot be set up in their place, but the writer believes that the case of the groined arch is too complicated for the development of theoretical formulas which can be even approximately correct, and suggests that practical experimenting and the development of empirical formulas is the only solution of such questions.

The writer has experimented on models of groined vaulting constructed of neat Portland cement on a scale of $\frac{1}{12}$ of an actual structure and presents in Plate IX, two photographs of one model showing the method of failure, which was nearly identical in all the models. In Plate IX, Fig. 1 shows the top and Fig. 2 the underside of the model.

The cracks are numbered in the order in which they appeared as the loading was applied, and it will be seen that the first six were tension cracks, and, furthermore, that all units indicated tension cracks before the compression cracks appeared on that unit.

The surprising thing about these models was, that they sustained far heavier loading before showing failure than would be supposed possible, and thus indicated that the accepted sections for such vaulting can be reduced materially, but, of course, experiments should be tried on full-sized sections before any general deductions can be made.

It can be seen also that if iron in tension is used in the proper place in a groined arch, the strength of the arch will be increased materially, and, conversely, that its section and cost for a given strength can be reduced materially.

A little time and money devoted to experimenting along the lines

Mr. Fuller. indicated would put us in possession of a light, durable and comparatively inexpensive covering for reservoirs and filter beds.

Concerning the calculation of quantities of the masonry vaulting the writer evolved formulas for the different cases some time ago, which gave the same results as those of the author, but, as his treatment of the subject differs from that of the author, a solution of one general case is appended to illustrate the difference in methods.

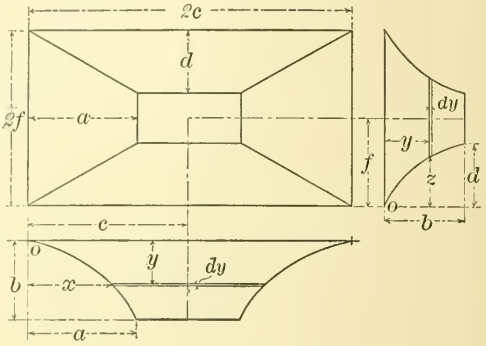

FIG. 17.

Take the case of the parabolic groined arch unit as shown in Fig. 17, and the following notation :

	Longitudinal Span.	Cross Span.
One-half distance center to center of piers...	c	f
One-half clear span.....	a	d
Rise of span.....	b	b

With origin at O the equations of the generating parabolas are

$$x = \sqrt{\frac{a^2}{b}}\, y,\ z = \sqrt{\frac{d^2}{b}}\, y$$

and

$$c - x = c - \sqrt{\frac{a^2\, y}{b}},\ f - z = f - \sqrt{\frac{d^2\, y}{b}}$$

The area of any dy plane above is $4\,(c - x)\,(f - z)$. The volume of any portion of the solid is

$$V = \int_{y_1}^{y_2} 4 \left(c - \sqrt{\frac{a^2\, y}{b}} \right) \left(f - \sqrt{\frac{d^2\, y}{b}} \right) dy$$

$$= 4 \left[c f\, y - \frac{2\, y^{\frac{3}{2}}}{3\sqrt{b}}\,(a f + c\, d) + \frac{a\, d\, y^2}{2\, b} \right] \Bigg|_{y_1}^{y_2}$$

with limits of b and o.

$$V = 4\, b \left[c f - \frac{2}{3}\,(a f + c\, d) + \frac{a\, d}{2} \right] \ldots\ldots\ldots\ldots\ (1)$$

The volume of the air space remaining, between the plane of springing lines and soffit of arch, is

$$v = 4\, b\, c f - V = \frac{2\, b}{3}\,(4\, a f + 4\, c\, d - 3\, a\, d) \ldots\ldots\ldots\ (2)$$

The value of v in formula (2) is the same as that found by the author after the necessary changes have been made in the notation.

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COMPARISON OF WEIGHTS OF A THREE-HINGED AND A TWO-HINGED SPANDREL-BRACED PARABOLIC ARCH.

Discussion.*

By HENRY S. JACOBY, Assoc. Am. Soc. C. E.

HENRY S. JACOBY, Assoc. Am. Soc. C. E. (by letter).—Since the adoption of the design of the spandrel-braced steel railway arch over the Niagara River, and the beginning of its construction, the writer has taken a renewed and more definite interest in the comparative study of different types of metallic arches. At his suggestion, two graduate students in the College of Civil Engineering of Cornell University last year made the computations necessary to determine the relative weight of three-hinged and two-hinged spandrel-braced arches having the same general dimensions and loading as the Niagara Railway Arch. Mr. George G. Smith, Jr., made the design for the two-hinged arch, while Charles C. More, Jun. Am. Soc. C. E., made the design for the three-hinged arch as well as that of a combination-type arch, which will be described in the latter part of this discussion. Mr. Jacoby.

The general dimensions, loading and specified unit stresses are given in the paper on "The Niagara Railway Arch†."

* This discussion (of the paper by C. W. Hudson, Assoc. M Am. Soc. C. E., printed in *Proceedings* for August, 1899) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 171.)

Communications on this subject received prior to November 24th, 1899, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

† By R. S. Buck, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xl, p. 125.

Mr. Jacoby. In finding the stresses in the trusses, however, two excess panel loads were substituted for the excess of the two locomotives above the corresponding weight of the train.

The Two-Hinged Arch.—In view of the statement made on page 137 of that paper, the given sectional areas of the members of the arch were not used except to find the stresses adopted in re-designing the sections. After this an additional revision was made, in order to see whether the final sections should generally depend upon the second or the third series of stresses, the first series being the preliminary ones which depend upon an assumption of equal sectional areas for all the members.

It was found that the required areas of the upper chord were increased from 0.5 to 1.1%, except those of U_7 and U_8 in Fig. 7, in which the increase was 2.9 and 3.3%, respectively. The areas of L_6 and L_7 were increased 0.5%, while those of the remaining lower-chord members were diminished from 0.4 to 1.3 per cent. The areas of the diagonals were increased from 0.2 to 1.8%, except that of D_7, in which the increase was 6.8 per cent. In the verticals, the increase of section varied from 1.2 to 3.5%, except for V_7, in which it was 6.8 per cent. Since most of these differences were covered by the excess of the sections adopted over those required, only a few members were affected by this revision.

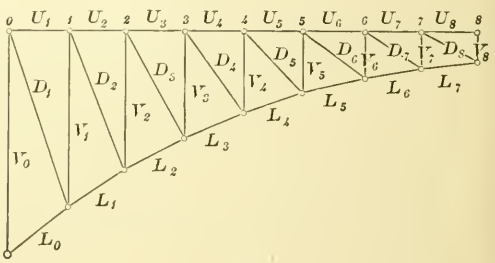

Fig. 7.

These changes in section are relatively much smaller than one is led to expect from the corresponding differences in the stresses, the result being due to the influence of the reverse stresses in the design. The temperature stresses were derived for a range extending from 75° above to 75° below the standard.

The above results indicate that the desirability of making the second revision depends largely upon the magnitude of the structure, while it is evidently affected to some extent by the form and proportion of the trusses.

The Three-Hinged Arch.—The dead-load stresses for the three-hinged arch were first found by using the same panel loads as those which had been determined from the design of the two-hinged arch. After this preliminary design was completed the dead panel loads were computed and found to exceed the previous values by percentages varying from 0.8 near the middle to 3.6 near the ends of the truss, while the middle panel load was 0.3% less. The average excess is 2.32 per cent.

Mr. Jacoby.

The dead-load stresses were revised accordingly, and the following differences were found, expressed as percentages of the sum of the dead, live and 0.8 times the reverse stresses. For the upper chord, 0.8 — 1.9; for the lower chord, 1.3 — 2.0; for the diagonals, 0.3 — 2.0; and for the verticals, 0.2 — 2.4. All the stresses were increased, except those in D_4, V_6, V_7 and V_8.

These changes, however, affected the sectional areas of only four members, the rest being covered by the small excesses in area made necessary by the make-up of the sections.

Comparative Weights.—Excluding the connecting plates, rivet heads and some minor details, all of which may be reasonably assumed to vary in the same ratio as the members of the respective trusses, it was found that the trusses of the three-hinged arch were 0.84% lighter than those of the two-hinged arch. The material is distributed as shown in Table No. 3.

TABLE No. 3.

	Upper chords.	Lower chords.	Diagonals.	Verticals.
Two-hinged arch.....	15.6%	36.7%	22.2%	25.5%
Three-hinged arch.....	12.9%	43.4%	20.1%	23.6%

It will be observed that in the three-hinged arch a considerably larger proportion of the material is contained in the lower chord, while the other classes of members contain less material than the two-hinged arch. Only four members in these classes, U_3, U_4, D_1 and D_2 are larger in the three-hinged arch. It may be of interest to note that the range of stress in the lower chord of the three-hinged arch is 6.5% less, while the maximum stress (excluding the wind) averages 17.0% more, and the live-load stress 26.8% more, than in the two-hinged arch.

In order that the influence of the crown hinge might be obscured as little as possible by extraneous conditions, the designs of the corresponding members in the two hinged and three-hinged arches were made as nearly alike in every respect as the required sectional areas would permit, the differences in the composition or make-up of the sections from that used in the construction of the Niagara Railway Arch, being reduced to a minimum. Since the distance from back to back of the angles in the chords was made the same in all cases, the additional material required in the lower chord of the three-hinged arch had generally to be placed on the inside and in the web plates, the result being to reduce the squares of the radii of gyration by an average of 4 per cent. This element places the three-hinged arch at a slight disadvantage.

Mr. Jacoby.

Comparison with the Arch of 200-Ft. Span.—The corresponding distribution of material in the trusses described by the author is given in Table No. 4.

TABLE No. 4.

	Upper chord.	Lower chord.	Diagonals.	Verticals.
Two-hinged arch	23.2%	39.6%	17.4%	19.7%
Three-hinged arch	17.7%	45.1%	19.4%	17.7%

The web members are relatively lighter than in Table No. 3, while the chords are heavier, notably the upper chord. Some of this disparity may be accounted for by the difference in the working stresses, as well as the differences in the relative loading and in the proportions of the trusses. The uniform live load in the Niagara Arch is 2.3 times as great, while the dead load is only 1.8 times as great per linear foot as in this arch. Again, the span is 2.75 times as great, while the depth at the crown is 3.33 times as great. The ratio of the rise of the lower chord to the span is very nearly the same. The trusses of the Niagara Arch are about 10 times as heavy as those of the smaller arch. The range of temperature was assumed as 25% greater. The differences between the required sectional areas and those adopted in the design are relatively much larger in the smaller arch on account of the smaller sections, and the greater influence of the limitations imposed by the minimum thickness of metal allowed.

The different conditions just stated likewise affect the relative weights of the trusses of the two types of arches, but it would require additional investigation to determine the magnitude of this effect.

One other element should be mentioned, namely, the effect of the wind stresses upon the truss members. For the comparative designs of the Niagara Arch it was assumed that no section should be increased in area unless the wind stress exceeded 40% of the sum of the dead, live and temperature stresses. It was found that this required an increase in only two chord members near the middle in the two-hinged arch, and one in the three-hinged arch.

It may, perhaps, be well to call attention to the fact that if in the smaller arch described in the paper the weight of the details be excluded, the difference in favor of the two-hinged arch is reduced to 3.9%, while if the details are included and made the same in both cases the differences will be only 2.7 per cent.

A Combination-Type Arch.—Any slight yielding in the foundations of a two-hinged arch materially changes the stresses, especially near the crown, while inaccuracies in the construction, in locating the end hinges, or in adjustment in closing the arch, have a similar effect. In

view of these facts, it appeared to the writer that it would be desirable to erect an arch with three hinges, and, after completion, when all the dead load is in place, to transform it into a two-hinged arch by connecting, at the standard temperature, the upper chord at the center and by riveting the connection at the crown hinge. Such an arrangement eliminates the effect of any inaccuracies in construction or erection, as well as those of the initial set of the abutments, due to the imposed loads. The resulting stresses are a combination of the dead-load stresses of a three-hinged arch with the live-load and temperature stresses of a two-hinged arch. Mr. Jacoby.

It seemed worth while to determine how the weight of such an arch would compare with the other two types, and whether it would possess any other merit than that of realizing more perfectly in construction the conditions which are assumed in its design than is done by the two-hinged arch.

In the first determination of the sectional areas the stresses obtained in the preceding designs were used, and then a revision was made with the aid of a new set of live and temperature stresses. The temperature stresses were $10\frac{3}{4}\%$ greater than before. In this revision the required sectional areas were increased, with three exceptions, by from 0.4 to 8.2%, the difference being below 2% in all but a few members. One-half of the areas at first adopted were increased by amounts varying from 1.0 to 3.8%, except one which had to be enlarged 5.9 per cent.

The weight of this truss, exclusive of the connecting details, was found to be 4.2% greater than that of the two-hinged arch. This excess in weight is due, mainly, to the influence of the larger reverse stresses in many of the members. The distribution of weight is as follows: Upper chords, 16.5%; lower chords, 37.2%; diagonals, 22.3%; and verticals, 24.0 per cent.

Comparing the various classes of members with those of the two-hinged arch, the results are: Upper chords, 9.7% heavier; lower chords, 5.8% heavier; diagonals, 4.8% heavier; and verticals, 1.9% lighter. With the three-hinged arch as a basis of comparison, the upper chord is 34.0% heavier; the lower chord, 9.8% lighter; the diagonals, 16.3% heavier; and the verticals, 7.0% heavier.

The distribution of the material in this combination-type of arch is such that it forms a stiffer structure than the two-hinged arch, its static deflection at the center under full live-load being nearly 10% less.

The writer wishes to express his appreciation of the great care exercised by Messrs. More and Smith in making the computations and graphic constructions involved in these designs, and from which this discussion has been prepared.

In conclusion, a note may be added in the interest of diminishing the labor required to find the stresses. If the stresses in the upper

Mr. Jacoby. chord due to $H = 1$ and $V = 1$ be computed and laid off on the horizontal lines in Figs. 3 and 6, respectively, experience shows that the rest of the stresses may be determined graphically with a precision that answers fully all the requirements of design, it being understood that a suitable scale is adopted. This procedure will save the tedious computations for the lengths of the lever arms of the remaining truss members.

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WHEEL CONCENTRATIONS AND FATIGUE FORMULAS IN BRIDGE DESIGN.

Discussion.*

By Messrs. O. E. SELBY, HENRY S. PRICHARD and CHARLES B. WING.

Mr. Selby. O. E. SELBY, Jun. Am. Soc. C. E. (by letter).—It is the purpose of the writer to present, first, three tables giving actual moments and shears in trusses and plate-girders produced by the loading in "Cooper's 1896 Specifications;" second, a table of the single-excess loads which, with the constant uniform train-load, will produce the same moments and shears as the wheel-loads; third, two diagrams on which these excess loads are plotted in order to determine their laws of variation; fourth, three simple formulas deduced from the diagrams, which will give values of excess loads producing moments and shears substantially equal to those produced by the wheel loads; fifth, a simple specification for loading which will combine in one operation the computing of the quiescent stresses and the allowance for impact.

The tables for moments and shears for Cooper's "$E40$" have been computed systematically and checked carefully, and will be useful to those who still prefer to, or find it necessary to, compute stresses by the actual wheel-loads for this specification.

Table No. 10 gives the moment, end shear, center shear and shear at quarter point for plate-girder spans, varying by increments of 1 ft., from 10 to 125 ft. Tables Nos. 11 and 12, respectively, give moments and shears for truss spans of 5, 6, 7, 8 and 9 panels, and panel lengths varying by increments of 1 ft. from 20 to 35 ft. This includes span lengths between 100 ft. and 315 ft. The shears apply only to trusses

* Continued from August, 1899, *Proceedings*.

Mr. Selby. with parallel chords. The arrangement of the tabulation by panel lengths rather than span lengths is believed to be new, and facilitates a complete tabulation in a small space. Values for intermediate panel lengths can be obtained by direct interpolation. All values in the tables are for one truss or girder.

Having prepared the foregoing tables for his own use, some time ago, the writer began to cast about for a simple substitute loading which could be used instead of wheel loads in general without material variation in the results, and for some rule for varying the substitute loading to correspond with different engine loadings. He made many comparative computations, using various single excesses, double excesses, uniform loads, etc., with results in favor of the single-excess method, but without finding any one excess load which gave satisfactory results for widely different span lengths. Finally, it occurred to him to compute, for the various positions and span lengths, the excess loads which, with the uniform train-load, would produce the actual moments and shears in the table, and to study their laws of variation.

Table No. 13 gives the values of such excess loads E, for one truss or girder, for all panel points of trusses of 5, 6, 7, 8 and 9 panels, with panel lengths varying by 3 ft. from 20 to 35 ft., and for plate-girders of lengths varying by 5 ft. from 10 to 125 ft.

The formulas for computing E are very simple. Let M = total actual moment, M_u = moment from uniform load and M_e = moment from excess load. Then $M = M_u + M_e$. Similarly for shears $S = S_u + S_e$.

Let w = uniform train-load per foot and l = span.

For moment in a plate-girder:

$$M = \frac{1}{8} w l^2 + \frac{1}{4} E l, \text{ from which}$$

$$E = \frac{4 M}{l} - \frac{w}{2} l.$$

For a train-load of 4 000 lbs. per foot, with 1 000-lb. units, $w = 2$ for one girder, then $E = \frac{4 M}{l} - l.$

For end shear in a plate-girder:

$$S = \frac{1}{2} w l + E, \text{ from which } E = S - \frac{1}{2} w l.$$

If $w = 2$, $E = S - l$.

For center shear in a plate-girder:

$$S = \frac{1}{4} w \frac{l}{2} + \frac{E}{2} = \frac{1}{8} w l + \frac{E}{2}, \text{ from which}$$

$$E = 2 S - \frac{1}{4} w l.$$

If $w = 2$, $E = 2 S - \frac{1}{2} l$.

Mr. Selby.

For moments in a truss, let N = whole number of panels and n = number of panels to the left of the point in question; then, considering the uniform panel load concentrated at the panel point,

$$M_u = \frac{N-1}{2} p\, w \times n\, p - \frac{n\,(n-1)}{2} p \times p\, w = p^2 \frac{w}{2} n\,(N-n).$$

If $w = 2$, $M_u = p^2\, n\,(N-n)$.

$$M_e = \frac{N-n}{N} E\, n\, p$$

$$M = p^2\, n\,(N-n) + \frac{N-n}{N} E\, n\, p = n\,(N-n)\, p\left(p + \frac{E}{N}\right), \text{ from which}$$

$$E = N\left(\frac{M}{n\,(N-n)\,p} - p\right)$$

For shears in a truss:

$$S_u = p\, w \frac{(N-n+1)\,(N-n)}{2N}.$$

If $w = 2$, $S_u = p \frac{(N-n+1)\,(N-n)}{N}$.

$$S_e = \frac{N-n}{N} E.$$

$$S = p \frac{(N-n+1)\,(N-n)}{N} + \frac{N-n}{N} E, \text{ from which}$$

$$E = \frac{N}{N-n} S - p\,(N-n+1).$$

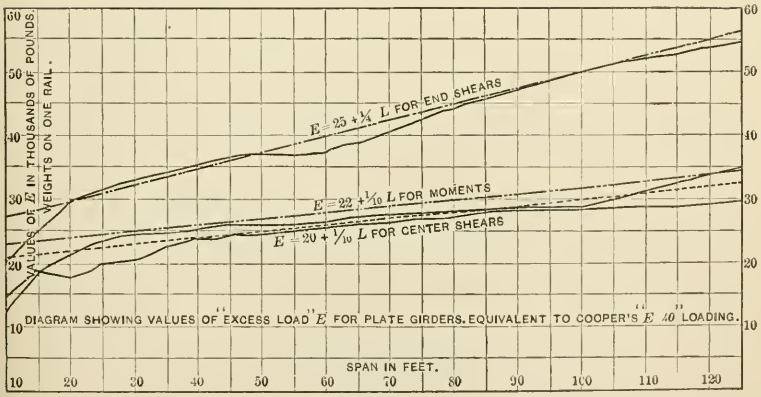


Fig. 8.

An inspection of Table No. 13 shows that the values of E increase slowly with the span length for plate-girders and are nearly uniform for different panel lengths, but vary with the position in the truss and the number of panels. The values for plate-girders have been plotted on the diagram, Fig. 8, together with straight lines, which coincide

Mr. Selby. practically with the curves of actual values. The equations of the straight lines are as follows:

For end shears	E, for one girder,	$=$	$25 + \frac{1}{4} L.$
" moments	"	"	"	$22 + \frac{1}{10} L.$
" center shears	"	"	"	$20 + \frac{1}{10} L.$

Shears and moments computed from values of E obtained by the foregoing formulas will differ from the actual values by very small percentages on the safe side, except for span lengths of less than 20 ft. The greatest differences are as follows: For moments—10-ft. span, 46.6%, 15-ft. span, 15.5%, 20-ft. span, 6.6%, 100-ft. span, 2.5%. For end shears—10-ft. span, 25%, 15-ft. span, 9.4%, 60-ft. span, 2.5%. For center shears—10-ft. span, 30%, 15-ft. span, 9.1%, 20-ft. span, 14.3%, 25-ft. span, 8%, 30-ft. span, 7.4%, 125-ft. span, 3.3 per cent.

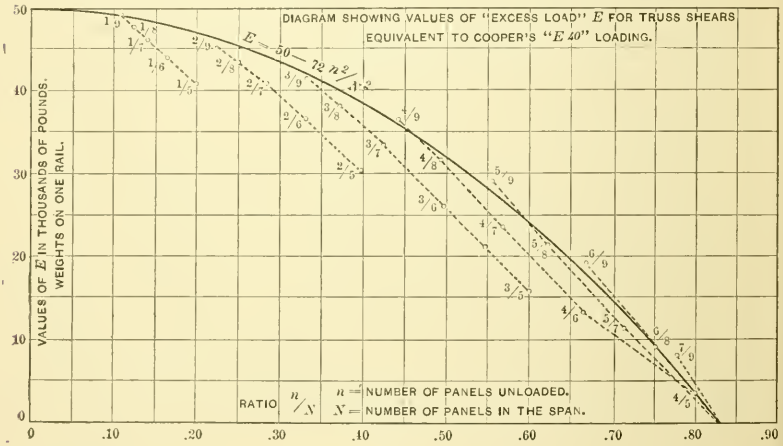


FIG. 9.

By the use of the above formulas for E, the greatest excess over the actual quiescent stress comes, as it should, on the shorter spans, where extra metal in a girder is desirable for the sake of mass. In a specification for actual use the number of formulas could be reduced to two, with advantage, using

$$E = 25 + \tfrac{1}{4} L \text{ for end shears, and}$$
$$E = 22 + \tfrac{1}{10} L \text{ for moments and center shears.}$$

The increase in values of center shears would not result in much increase in the weight of metal.

The values of E for truss shears have been plotted on the diagram, Fig. 9, taking the average value at each panel point for the different panel lengths. The abscissas of this diagram represent percentages of span unloaded. The parabola plotted will give values of E which will produce shears practically equal to the actual shears; the greatest

excesses being on the short spans and on counter shears, where they should be. The greatest percentages of difference are as follows: Mr. Selby.

$\frac{1}{6}$	20-ft. panel	7.7%
$\frac{2}{5}$	20 "	12.3%
$\frac{3}{5}$	20 "	8.4%
$\frac{3}{5}$	29 "	8.3%
$\frac{4}{6}$	20 "	3.6%
$\frac{4}{6}$	29 "	7.0%

The equation of the parabola is E, for one truss, $= 50 - 72 \frac{n^2}{N^2}$, in which n = number of panels unloaded, and N = whole number of panels. The values of E by this formula, for the different panel ratios, are shown in Table No. 14.

TABLE No. 14.

Panels unloaded.	Whole Number of Panels.				
	5	6	7	8	9
1	47	48	49	49	49
2	38	42	44	46	46
3	24	32	37	40	42
4	4	18	26	32	36
5	....	0	13	22	28
6	....	....	....	10	18
7	....	....	....	....	6

The use of this formula (or Table No. 14) for E, in a specification, would give shears substantially equal to, and never appreciably less than, those by Cooper's Specification.

The values of E for truss moments do not vary much below 40 for one truss. Using a constant value of 40, the greatest percentages of excess are as follows:

$\frac{2}{5}$	20-ft. panel	6.5%
$\frac{3}{6}$	20 " "	4.4%
$\frac{4}{8}$	32 " "	3.4%
$\frac{4}{8}$	35 " "	4.1%
$\frac{4}{9}$	32 " "	3.1%
$\frac{4}{9}$	35 " "	3.7%

An inspection of the diagrams, Figs. 8 and 9, shows that the method proposed by Mr. Cowles, of using the uniform train-load headed by the maximum floor-beam reaction, will generally give results too small. The panel excess derived from the floor-beam reactions for panels of 20 to 35 ft. varies from 25.6 to 27.5, and averages 26.5 for one truss.

It will be seen that the necessary excess is greater, and usually much greater, than this, except for counter shears and short plate-girders. The writer knows of one bridge now being built under

Mr. Selby. Cooper's Specifications, in which the equivalent loading is taken as 4 000 lbs. per foot, with 50 000 lbs. excess. The specifications of a prominent system call for two classes of loading: Class *A*, 5 000 lbs. per foot, with 50 000 lbs. excess; and Class *B*, 4 000 lbs. per foot, with 40 000 lbs. excess. It is evident from the diagrams that these values of 20 and 25 for *E* are too small to give consistently-designed bridges for a uniform train-load headed by two correspondingly heavy consolidation engines.

It may seem strange, at first glance, that the values of *E* should be smaller for short plate-girders than for longer ones, but it will be found that the value of *E* divided by the span length is greater for the short spans than for the long ones; that is, the "equivalent uniform load," increases as the span decreases.

A specification based on the foregoing formulas, viz., the use of a uniform load of 2 000 lbs. per foot for one truss or girder with a single excess, determined by—

$E = 40$ for truss moments,

$E = 50 - 72 \frac{n^2}{N^2}$ " " shears,

$E = 25 + \frac{1}{4} L$ " plate-girder end shears, and

$E = 22 + \frac{1}{10} L$ " " " moments and center shears,

would surely give a consistently proportioned structure, with stresses as nearly equivalent to the actual quiescent ones as it is possible to obtain by simple methods. The uniform load is constant, and the values of *E* are determined by easily memorized formulas, no tables or diagrams being necessary. Mr. Waddell's "equivalent uniform load" method is necessarily cumbersome, because the load for any given span has to be taken from a diagram plotted from computed equivalents, and cannot be memorized. The diagram must be before one when beginning the computation of a span. Mr. Waddell, in his discussion, says: "In these tables the best possible showing was made for both the single-concentration method and the double-concentration method." It is evident that it is not the best possible showing for the single-excess method to compare results obtained by using a constant value of *E* with those obtained by using a variable value of uniform load computed for the cases compared. To illustrate by an extreme case, if the variable excesses from Table No. 13 were used in a comparison, the showing for the single-excess method would be very good, indeed, as the percentage of error would be zero in every case. It has been shown that the differences from using the foregoing simplified formulas are very small, except for spans less than 20 ft., and that, in all cases, the excess of metal resulting is placed where it will do the most good.

Finally, by introducing the consideration of impact, and, bearing in mind the widely varying ideas as to the proper allowances therefor, it becomes possible to simplify still further the specified loading, and

to compute at one operation both the quiescent stresses and the impact allowance. Tables Nos. 15, 16 and 17 show the percentage of excess stress from using a constant value of E for all purposes, and a comparison with the allowance for impact specified by the railroad system with which the writer is connected. The formula for impact is $I = \frac{10}{L + 10}$, in which L is the span length for plate-girders and for truss moments, and is the length of span loaded for truss shears. Mr. Selby.

TABLE No. 15.—Plate-Girder Spans.

	25 ft.	50 ft.	75 ft.	100 ft.	125 ft.
Moments $E = 50$	54%	31 %	22 %	16%	9.5%
" $E = 40$	33%	18.5%	12 %	9%	3.2%
End Shears $E = 50$	33%	15 %	6.2%	0%	0 %
Impact $I = \frac{10}{L+10}$	30%	16.7%	12 %	9%	7.5%

TABLE No. 16.—Truss-Center Moments, 20-Ft. Panels.

	5 panels.	6 panels.	7 panels.	8 panels.	9 panels.
$E = 50$	14%	10 %	6.5%	5.6%	5 %
Impact $I = \frac{10}{L+10}$	9%	7.7%	6.7%	6 %	5.3%

TABLE No. 17.—Truss Shears. 9 Panels at 20 ft.

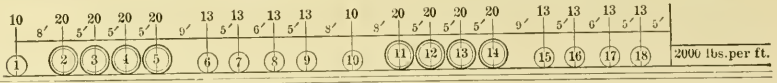
Panel points	1.	2.	3.	4.	5.	6.	7.
$E = 50$	0.6%	2.3%	5.0%	10%	19%	34%	58.5%
Impact $I = \frac{10}{L+10}$	6.0%	6.7%	7.7%	9%	11%	14%	20.0%

Ideas vary so much as to the correct allowance for impact, that the above formulas will answer for comparison as well as any, and the correspondence is close enough to make it plain that the use of a constant single-excess load for all spans, except plate-girder spans under 20 ft., comes very near to giving, with one operation, the desired result, namely, the actual stress plus the impact.

The writer would, therefore, favor, for a standard railroad bridge specification, the use of a uniform load of 4 000 lbs. per foot of track, together with a single excess of 100 000 lbs., placed at the most effective point, with no other allowance for impact for all spans over about 20 ft. For spans under 20 ft. stresses would be determined by a series of the heaviest passenger-axle loads with an impact allowance of $I = \frac{10}{L + 10}$.

TABLE No. 10.—Maximum Moments and Shears in Plate Girders. Cooper's 1896 Specifications "E 40."

Moments in Thousands of Ft.-Lbs. Shears in Thousands of Lbs. Weights on One Rail.

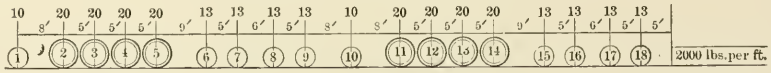


Span.	Moment.	Shears. End.	Shears. ¼ Pt.	Shears. Center.	Span.	Moment.	Shears. End.	Shears. ¼ Pt.	Shears. Center.	Span.	Moment.	Shears. End.	Shears. ¼ Pt.	Shears. Center.	Span.	Moment.	Shears. End.	Shears. ¼ Pt.	Shears. Center.
.....	.....	.....	.....	.....	36.....	548.6	70.6	44.1	20.6	66.....	1 539.5	105.0	63.6	29.7	96.....	2 993.9	144.8	83.3	38.3
.....	.....	.....	.....	.....	37.....	574.3	71.9	44.8	21.0	67.....	1 581.5	106.4	64.2	30.0	97.....	3 049.0	146.2	84.2	38.5
8.....	37.8	27.5	17.5	10.0	38.....	600 0	73.1	45.4	21.3	68.....	1 623.5	107.8	64.8	30.2	98.....	3 106.5	147.5	85.0	38.8
9.....	46.9	28.9	18.9	10.0	39.....	626.6	74.3	46.0	21.7	69.....	1 665.5	109.2	65.4	30.5	99.....	3 162.3	148.8	85.8	39.1
10.....	56.3	30.0	20.0	10.0	40.....	655.6	75.4	46.8	22.0	70.....	1 706.9	110.5	65.9	30.7	100.....	3 219.9	150.0	86.6	39.4
11.....	65.7	32.7	20.9	10.9	41.....	684.6	76.8	47.5	22.3	71.....	1 749.3	111.8	66.5	31.1	101.....	3 277.6	151.2	87.3	39.6
12.....	80.0	35.0	21.7	11.7	42.....	713.6	78.1	48.2	22.6	72.....	1 792.2	113.3	67.0	31.4	102.....	3 335.9	152.4	88.1	39.9
13.....	95.0	36.9	22.3	12.3	43.....	742.6	79.4	48.9	22.9	73.....	1 833.9	114.8	67.5	31.7	103.....	3 410.6	153.7	88.8	40.1
14.....	110.0	38.6	23.6	12.9	44.....	771.6	80.6	49.5	23.2	74.....	1 879.2	116.3	68.0	32.0	104.....	3 475.2	154.9	89.5	40.4
15.....	125.0	40.0	25.0	13.3	45.....	800.6	81.7	50.1	23.4	75.....	1 925.8	117.7	68.6	32.3	105.....	3 537.6	156.1	90.3	40.6
16.....	140.0	42.5	26.3	13.8	46.....	829.6	82.8	50.7	23.7	76.....	1 972.0	119.1	69.2	32.6	106.....	3 600.3	157.3	90.9	40.9
17.....	155.0	44.7	27.4	13.8	47.....	858.6	83.8	51.4	23.9	77.....	2 019.1	120.4	69.9	32.9	107.....	3 666.6	158.5	91.7	41.1
18.....	170.0	46.7	28.3	13.9	48.....	887.6	84.8	52.1	24.2	78.....	2 064.4	121.7	70.5	33.2	108.....	3 745.3	159.6	92.4	41.3
19.....	186.6	48.4	29.2	14.0	49.....	918.8	86.0	52.8	24.5	79.....	2 112.3	123.0	71.1	33.4	109.....	3 818.4	160.8	93.2	41.6
20.....	206.3	50.0	30.0	14.0	50.....	950.9	87.1	53.5	24.9	80.....	2 160.5	124.2	71.7	33.7	110.....	3 886.8	162.0	93.9	41.8
21.....	226.0	51.4	31.4	14.5	51.....	983.1	88.2	54.1	25.2	81.....	2 207.7	125.6	72.3	34.0	111.....	3 958.2	163.2	94.6	42.0
22.....	245.7	52.7	32.7	15.0	52.....	1 015.2	89.2	54.8	25.5	82.....	2 256.7	126.9	73.0	34.4	112.....	4 026.9	164.4	95.3	42.2
23.....	265.4	53.9	33.9	15.4	53.....	1 047.4	90.2	55.4	25.8	83.....	2 306.5	128.2	73.7	34.7	113.....	4 099.0	165.5	96.0	42.5
24.....	285.2	55.0	35.6	15.8	54.....	1 081.4	91.2	56.1	26.1	84.....	2 356.3	129.5	74.4	35.0	114.....	4 172.0	166.7	96.8	42.8
25.....	305.0	56.5	36.0	16.2	55.....	1 116.9	92.1	56.8	26.4	85.....	2 406.9	130.7	75.1	35.3	115.....	4 245.0	167.9	97.5	43.1
26.....	324.8	57.9	36.9	16.5	56.....	1 152.4	93.0	57.5	26.6	86.....	2 459.6	132.1	75.8	35.6	116.....	4 318.8	169.0	98.3	43.4
27.....	344.6	59.2	37.8	16.9	57.....	1 187.9	94.2	58.2	26.9	87.....	2 510.6	133.4	76.5	35.9	117.....	4 389.5	170.2	99.0	43.7
28.....	365.5	60.4	38.6	17.1	58.....	1 223.4	95.4	58.8	27.2	88.....	2 564.2	134.7	77.1	36.2	118.....	4 463.8	171.4	99.7	43.9
29.....	388.0	61.6	39.3	17.4	59.....	1 261.0	96.5	59.5	27.5	89.....	2 615.9	136.0	77.9	36.5	119.....	4 538.8	172.5	100.4	44.2
30.....	410.5	63.0	40.0	17.7	60.....	1 299.6	97.6	60.1	27.9	90.....	2 670.5	137.2	78.7	36.7	120.....	4 614.1	173.7	101.1	44.5
31.....	432.9	64.4	40.7	18.2	61.....	1 338.3	98.7	60.7	28.2	91.....	2 723.0	138.5	79.5	37.0	121.....	4 686.5	174.8	101.8	44.7
32.....	455.4	65.7	41.3	18.8	62.....	1 377.0	100.0	61.3	28.5	92.....	2 775.9	139.8	80.3	37.3	122.....	4 762.7	176.0	102.5	45.0
33.....	477.9	66.9	42.0	19.2	63.....	1 415.6	101.3	61.8	28.8	93.....	2 831.5	141.1	81.0	37.5	123.....	4 836.2	177.1	103.2	45.3
34.....	500.4	68.1	42.8	19.7	64.....	1 455.5	102.6	62.4	29.1	94.....	2 885.3	142.4	81.7	37.8	124.....	4 917.4	178.3	104.0	45.7
35.....	523.0	69.2	43.5	20.1	65.....	1 497.5	103.8	63.0	29.4	95.....	2 939.5	143.6	82.5	38.0	125.....	4 996.4	179.4	104.7	46.0

Mr. Selby.

Mr. Selby.

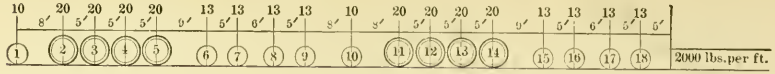
TABLE No. 11.—Table of Maximum Moments. Cooper's 1896 Specification "E 40." Moments in Thousands of Foot Pounds. Weights on One Rail.



Panel Points.	Panel Lengths.																
	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	
1/5	2 186.2	2 400.4	2 622.4	2 849.8	3 081.8	3 320.2	3 565.0	3 816.2	4 073.8	4 337.8	4 608.2	4 890.0	5 181.2	5 478.8	5 782.8	6 093.2	
1/6	2 689.3	2 942.7	3 206.8	3 479.3	3 760.2	4 049.3	4 346.8	4 652.7	4 966.8	5 289.3	5 620.2	5 959.3	6 312.7	6 676.8	7 049.3	7 430.2	
1/7	3 163.7	3 453.4	3 761.3	4 080.1	4 409.3	4 748.7	5 098.4	5 458.4	5 828 7	6 209.3	6 600.1	7 001.3	7 413.1	7 843.7	8 284.3	8 735.1	
1/8	3 619.5	3 951.1	4 298.1	4 663.0	5 040.1	5 429.5	5 831.1	6 245.0	6 671.1	7 109.5	7 560.1	8 023.0	8 498.1	8 991.1	9 499.5	10 020.1	
1/9	4 062.9	4 436.2	4 823.8	5 238.9	5 658.8	6 097.9	6 551.2	7 018.8	7 500.6	7 996.6	8 506.8	9 031.2	9 569.9	10 125.6	10 701.6	11 291.8	
2/5	3 154.4	3 477.6	3 813.6	4 154.4	4 512.0	4 874.4	5 241.6	5 613.6	5 993.4	6 387.0	6 785.4	7 188.6	7 596.6	8 009.4	8 472.4	8 953.6	
2/6	4 204.7	4 599.3	4 999.3	5 404.7	5 815.3	6 234.7	6 682.3	7 167.7	7 663.7	8 173.7	8 704.3	9 245.7	9 797.7	10 381.7	10 981.7	11 608.3	
2/7	5 060.6	5 531.4	6 024.3	6 537.9	7 073.6	7 623.6	8 187.9	8 783.6	9 393.6	10 017.9	10 664.3	11 331.4	12 012.9	12 715.4	13 448.3	14 204.6	
2/8	5 966.3	6 530.3	7 112.3	7 712.3	8 344.0	8 995.0	9 664.0	10 360.0	11 083.0	11 824.0	12 584.3	13 376.3	14 186.3	15 014.3	15 880.3	16 768.3	
2/9	6 862.4	7 503.1	8 173.6	8 865.8	9 588.1	10 339.4	11 112.6	11 910.1	12 745.4	13 602.6	14 481.4	15 393.8	16 331.8	17 291.6	18 285.6	19 313.8	
3/6	4 596.5	5 057.0	5 536.5	6 030.0	6 532.5	7 056.5	7 592.0	8 136.5	8 690.0	9 252.5	9 836.5	10 432.0	11 036.5	11 650.0	12 272.5	12 916.5	
3/7	6 117.1	6 677.7	7 262.9	7 861.1	8 487.3	9 155.9	9 850.3	10 563.4	11 290.3	12 030.9	12 787.3	13 572.4	14 371.3	15 183.9	16 010.1	16 859.4	
3/8	7 387.6	8 056.5	8 795.4	9 581.0	10 385.4	11 208.5	12 051.0	12 930.4	13 828.5	14 745.4	15 681.0	16 648.5	17 640.4	18 651.0	19 680.4	20 735.4	
3/9	8 704.3	9 520.3	10 381.7	11 293.7	12 245.7	13 221.7	11 221.7	15 257.3	16 325.3	17 417.3	18 533.3	19 681.7	20 865.7	22 073.7	23 305.7	24 566.7	
4/8	7 954.0	8 690.0	9 444.5	10 232.5	11 036.5	11 856.5	12 700.0	13 572.0	14 460.0	15 368.5	16 324.5	17 309.0	18 317.0	19 359.0	20 423.0	21 520.5	
4/9	9 738.2	10 647.1	11 578.2	12 547.1	13 544.9	14 564.9	15 611.6	16 698.2	17 807.1	18 942.2	20 135.6	21 363.9	22 623.9	23 926.1	25 257.2	26 631.1	

TABLE No. 12.—Table of Maximum Shears.

Cooper's 1896 Specification "E 40"
Shears in Thousands of Pounds.
Weights on One Rail.



Panel Points.	Panel Lengths.															
	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
1/5	109.3	114.3	119.2	123.9	128.4	132.8	137.1	141.3	145.5	149.6	153.6	157.7	161.9	166.0	170.1	174.1
1/6	134.5	140.1	145.8	151.3	156.7	162.0	167.2	172.3	177.4	182.4	187.3	192.2	197.3	202.3	207.3	212.3
1/7	158.2	164.4	171.0	177.4	183.7	189.9	196.1	202.2	208.2	214.1	220.0	225.8	231.7	237.7	243.7	249.6
1/8	181.0	188.1	195·4	202.7	210.0	217.2	224.3	231.3	238.3	245.2	252.0	258.8	265.6	272.5	279.4	286.3
1/9	208.1	211.2	219.3	227.6	235.8	243.9	252.0	260.0	267.9	275.7	283.6	291.3	299.1	306.8	314.8	322.6
2/5	63.3	66.1	68.9	71.9	74.8	77.3	80.1	82.9	85.7	88.5	91.2	93.8	96.3	98.8	101.2	103.6
2/6	88.8	92.8	96.6	100.3	104.1	107.6	111.2	114.8	118.4	121.9	125.3	128.7	132.1	135.4	138.7	141.9
2/7	113.6	118.3	122.9	127.4	131.8	136.2	140.7	145.2	149.6	154.0	158.3	162.6	166.8	171.0	175.1	179.2
2/8	137.0	142.5	148.0	153.3	158.6	163.9	169.3	174.7	180.0	185.3	190.5	195.7	200.8	205.9	211.0	216.0
2/9	159.6	166.0	172.4	178.6	184.8	191.0	197.3	203.6	209.9	216.1	222.2	228.3	234.4	240.4	246.4	252.4
3/5	31.0	32.2	33.3	34.3	35.4	36.4	37.5	58.6	39.7	41.0	42.2	43.4	44.7	45.9	47.3	48.6
3/6	50.8	53.3	55.7	58.3	60.7	62.9	65.2	67.4	69.6	71.7	73.9	76.0	78.0	80.0	82.0	84.1
3/7	74.5	78.0	81.3	84.6	87.8	90.9	93.9	96.9	99.8	102.6	105.5	108.2	111.1	114.0	116.8	119.7
3/8	98.0	102.1	106.2	110.2	114.1	118.0	121.8	125.6	129.3	132.9	136.6	140.3	144.1	147.8	151.5	155.1
3/9	120.5	125.5	130.4	135.2	140.0	144.6	149.3	153.9	158.4	163.0	167.6	172.2	176.8	181.4	186.0	190.5
4/5	8.8	9.2	9.6	10.0	10.3	10.7	11.1	11.5	11.8	12.1	12.5	12.8	13.1	13.4	13.7	14.0
4/6	25.1	26.2	27.2	28.0	28.8	29.7	30.5	31.3	32.0	32.8	33.9	34.9	36.1	37.1	38.3	39.4
4/7	42.7	44.4	46.2	48.5	50.7	52.6	54.6	56.6	58.5	60.3	62.2	64.1	65.9	67.6	69.3	71.0
4/8	63.8	66.9	69.8	72.8	75.6	78.4	81.1	83.7	86.3	88.8	91.3	93.8	96.2	98.7	101.0	103.4
4/9	85.8	89.6	93.3	96.9	100.4	103.9	107.3	110.7	114.0	117.3	120.5	123.8	127.0	130.1	133.3	136.4
5/6	6.7	7.1	7.4	7.8	8.1	8.4	8.8	9.1	9.4	9.6	10.0	10.3	10.5	10.8	11.1	11.3
5/7	21.0	21.9	22.8	23.5	24.2	25.0	25.7	26.4	27.0	27.7	28.5	29.2	30.0	30.8	31.9	32.8
5/8	36.9	38.4	29.9	41.7	43.3	45.1	46.7	48.4	50.1	51.8	53.5	55.1	56.7	58.3	59.8	61.3
5/9	56.1	58.6	61.1	63.6	66.2	68.6	71.1	73.4	75.8	78.1	80.3	82.5	84.7	86.9	89.1	91.2
6/7	5.1	5.5	5.8	6.1	6.4	6.8	7.1	7.3	7.6	7.9	8.1	8.4	8.7	8.9	9.1	9.4
6/8	17.9	18.7	19.5	20.2	20.8	21.5	22.1	22.7	23.3	23.9	24.6	25.2	26.0	26.6	27.4	28.1
6/9	32.3	33.7	35.1	36.6	38.1	39.7	41.2	42.6	44.0	45.4	46.7	48.2	49.6	51.0	52.4	53.7
7/8	4.0	4.3	4.7	4.9	5.2	5.5	5.8	6.1	6.3	6.5	6.8	7.0	7.3	7.5	7.7	7.9
7/9	15.4	16.2	16.9	17.5	18.1	18.7	19.3	19.9	20.3	21.0	21.5	22.2	22.8	23.4	24.1	24.7
8/9	3.1	3.4	3.7	4.0	4.3	4.5	4.8	5.1	5.3	5.5	5.7	6.0	6.2	6.4	6.6	6.8

TABLE No. 13.—Values of "Excess Load" E, which, with 2 000 Lbs. per Foot on One Rail, will give same Moments and Shears as Cooper's "E 40" Loading. Unit 1 000 lbs.

Trusses.														Plate Girders.			
Moments.							Shears.										
	Panel Lengths.							Panel Lengths.									
$\frac{n}{N}$	20	23	26	29	32	35	$\frac{n}{N}$	20	23	26	29	32	35	Span.	Moment.	End shear.	Center shear.
.....	.....	.....	.....	.....	.....	.....	1/5..	36.7	39.9	41.4	42.0	42.4	42.6	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	1/6..	41.3	43.5	44.6	44.9	44.7	44.8	10	12.5	20.0	15.0
.....	.....	.....	.....	.....	.....	.....	1/7..	44.5	46.0	46.8	46.4	46.3	46.2	15	18.3	25.0	19.1
.....	.....	.....	.....	.....	.....	.....	1/8..	46.8	47.7	48.3	48.2	47.5	47.2	20	21.3	30.0	18.0
.....	.....	.....	.....	.....	.....	.....	1/9..	48.5	49.0	49.5	49.2	48.4	48.0	25	23.8	31.5	19.9
2/5...	31.4	35.5	38.0	38.6	37.9	38.2	2/5..	25.5	27.8	29.5	31.5	32.5	32.7	30	24.7	33.0	20.4
2/6...	37.7	38.2	36.8	37.4	37.6	38.8	2/6..	33.2	35.5	36.8	37.9	38.2	37.9	35	24.8	34.2	22.7
2/7...	37.1	38.0	38.4	38.8	38.8	39.1	2/7..	39.0	40.4	41.0	41.6	41.5	40.9	40	25.6	35.4	24.0
2/8...	38.9	39.5	39.8	39.8	39.5	39.4	2/8..	42.7	43.4	43.7	44.1	43.7	43.0	45	26.2	36.7	24.3
2/9...	40.6	40.8	40.8	40.5	40.0	39.7	2/9..	45.2	45.6	45.7	45.8	45.4	44.5	50	26.1	37.1	24.8
.....	.....	.....	.....	.....	.....	.....	3/5..	17.5	16.8	15.8	15.5	15.8	16.5	55	26.2	37.1	25.3
3/6...	33.2	36.8	38.6	38.7	37.9	36.0	3/6..	21.6	24.6	26.4	27.4	28.0	28.2	60	26.6	37.6	25.8
3/7...	38.4	38.4	39.0	39.0	38.0	36.0	3/7..	30.4	33.1	34.1	34.6	34.4	34.5	65	27.2	38.8	26.3
3/8...	37.0	38.2	39.2	39.2	38.0	36.0	3/8..	36.8	38.3	38.9	38.6	38.6	38.2	70	27.5	40.5	26.4
3/9...	37.6	38.5	39.5	39.3	38.1	35.9	3/9..	40.8	41.8	42.0	41.5	41.2	40.8	75	27.7	42.7	27.1
.....	.....	.....	.....	.....	.....	.....	4/5..	4.0	4.0	3.5	2.5	1.5	0.0	80	28.0	44.2	27.1
.....	.....	.....	.....	.....	.....	.....	4/6..	15.3	15.0	13.5	11.4	12.3	13.2	85	28.3	45.7	28.1
.....	.....	.....	.....	.....	.....	.....	4/7..	19.6	21.2	23.4	24.7	25.8	25.7	90	28.7	47.2	28.4
4/8...	38.9	38.5	36.2	33.0	30.2	27.4	4/8..	27.6	30.6	32.2	32.6	32.4	31.8	95	28.8	48.6	28.5
4/9...	39.2	38.5	36.2	32.9	30.2	27.4	4/9..	34.4	36.4	37.1	37.1	36.6	35.5	100	28.8	50.0	28.8
.....	.....	.....	.....	.....	.....	.....	5/6..	0.2	0.8	0.8	−0.4	−1.0	−2.2	105	29.8	51.1	28.7
.....	.....	.....	.....	.....	.....	.....	5/7..	13.5	13.3	12.0	10.0	9.0	9.8	110	31.3	52.0	28.6
.....	.....	.....	.....	.....	.....	.....	5/8..	18.4	19.2	20.5	22.1	23.2	23.5	115	32.7	52.9	28.7
.....	.....	.....	.....	.....	.....	.....	5/9..	26.2	28.1	30.0	30.7	30.6	30.2	120	33.8	53.7	29.0
.....	.....	.....	.....	.....	.....	.....	6/8..	11.6	11.8	10.4	8.7	8.0	7.5	125	34.9	54.4	29.5
.....	.....	.....	.....	.....	.....	.....	6/9..	16.9	17.8	19.6	20.2	20.8	21.1	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	7/9..	9.3	9.8	8.9	7.5	6.6	6.2	.....	.....	.....	.....

Mr. Prichard. HENRY S. PRICHARD, M. Am. Soc. C. E. (by letter).—If the term Railroad Bridge is understood to include the track, then the loads to which railroad bridges are subjected are concentrated wheel-loads. These loads are somewhat distributed over the remainder of the structure by the track, which slightly lessens the stresses in the connections of stringers to floor beams and reduces very considerably the pressures on ties and on local points in the flanges of stringers, but, except in these regards, the stresses produced are the same as if the loads were concentrated directly on the flanges of the stringers.

The most accurate method of specifying the loading to which a bridge is subjected is, of course, that of specifying the actual loads. For present conditions, the critical loadings are approximately knowable, therefore, for present conditions, the most accurate method of specifying the loading is to specify wheel concentrations. If, in a bridge designed for present conditions, the loads increase, but still maintain the same relations between the various wheel concentrations of both the engines and the cars, then the live-load stresses will be increased in the same proportion, and if the bridge was originally well designed for the original wheel concentrations, the distribution of the material will remain the most economical for the increased loading. Even if the relations between the wheel concentrations should vary greatly in the future, in a manner not intentionally provided for, there is no reason to suppose that any method other than that of specifying wheel concentrations would be more accurate as regards these future conditions, except by mere chance. From the foregoing facts, it is evident that, as far as accuracy is concerned, there is no reason why wheel concentrations should not continue to be specified.

In the discussion* Mr. Scofield and Mr. Snow have pointed out, what few if any will deny, that to those, including many bridge buyers, who are not expert in bridge design, a definite engine-loading is more intelligible than methods of supposed equivalents.

In fact, the method of specifying wheel concentrations is the only one which those who are not experts can use understandingly to give a close equivalent for the traffic conditions they anticipate, unless they obtain the assistance of an expert. Even for experts, the easiest method of specifying loads which satisfy their ideas regarding the traffic conditions, is to specify the engine wheel-concentrations.

This is shown by the fact, well illustrated in the discussion, that nearly all who have proposed methods other than by wheel-concentrations, have arrived at the loads they specify by first deciding on certain engine wheel-concentrations and then, with more or less labor (generally considerable), determining on supposedly close equivalents.

From what has been stated, it follows that, as far as facility in

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writing specifications is concerned, there is no reason why the method of specifying wheel concentrations should not be continued. Mr. Prichard.

The questions of accuracy and facility of specifying the loads being decided in favor of the method of engine wheel-concentrations, the question of the wisdom of abandoning its use depends on the value of the advantage as to accuracy, compared with the increased cost, if any, by which this advantage is obtained. To those who are not familiar with the short ways which have been devised for determining the stresses from wheel concentrations, or who, from lack of practice, cannot use them with facility, the cost of making the calculations probably seems much greater than it really need be. For stringers, floor beams, hangers, plate-girders and single-intersection trusses, resting on two supports, the calculation of the stresses from concentrated wheel-loads by proficient specialists is a very simple matter; in fact, it takes less time than by using some of the other systems which have been proposed, and the cost is insignificant, as compared with the total cost of the bridge. Such cases include the great majority of railway bridge work, and for them there would seem to be little advantage in substituting for this method an approximately equivalent one, even if a satisfactory one is obtainable. Be this as it may, it is certain that before the method of wheel-concentrations can be abandoned, some other method or methods must be selected to take its place. From present appearances it would be a perfectly safe proposition for the advocates of this method to agree to abandon its use when its opponents unite in recommending some other to take its place.

For the trusses of long spans it is not especially difficult to obtain other methods which give results agreeing closely with those obtained from any system of wheel-concentrations of usual type, and a fair agreement can be obtained for trusses of spans of moderate length, but for short spans and for floor systems the writer knows of no other method which gives such close approximations in all cases. For multiple-intersection trusses, for continuous girders, and for some other special cases there is much more reason for adopting some approximately equivalent method, than there is in the cases previously discussed, because, for such cases, the use of the method of wheel-loads is often quite complicated, and in some instances it is extremely so. For such cases, even when wheel-loads are specified, approximately equivalent methods are frequently allowed and used.

There is one phase of the use of the method of wheel-concentrations which requires notice, because misapprehension regarding it has resulted in throwing not a little discredit upon it. The fact referred to is, that this method has a superficial appearance of much nearer approach to accuracy than it really possesses, owing to the uncertainty as to future traffic conditions, the impossibility of making

Mr. Prichard. accurate allowance for the dynamic effect of the load, and other pertinent considerations. This fact has made the method of wheel-concentrations a butt for ridicule. In as far as this ridicule is the means of disabusing the minds of those who would otherwise be deceived by the seeming accuracy of the method it is commendable, but to the considerable extent by which it discredits the real advantage as to the accuracy of this method, as compared with others, it is harmful. The thoughtful advocates of the use of the method of wheel-concentrations advocate its use in spite of, and not in consequence of, its seeming accuracy, because, notwithstanding the fact that it is really far from accurate, it approaches nearer to accuracy than any other method. If those who believe that this advantage as to accuracy is worth what it costs in time and trouble should refrain from using this method simply because it has a superficial appearance of accuracy which is deceptive, they would be allowing their judgment to be influenced by purely sentimental considerations.

While the fact that it is not possible to obtain the stresses from the live load accurately is not a sufficient warrant for abandoning the use of the method of wheel-concentrations, it affords an excellent reason for departing from the exact weights and spacings of the wheel-concentrations and substituting therefor some simplified typical engine.

Mr. Wing. CHARLES B. WING, Assoc. M. Am. Soc. C. E. (by letter).—With our present limited knowledge of the stresses in bridge superstructures, due to rapidly moving train loads, any refinement of calculation of stresses from assumed loads is not justified, if the cost of the structure is thereby increased. To quote an axiom familiar to bridge engineers: "Stresses should not be figured to half a pound when you guess at the load."

In discussing the advisability of discontinuing the use of the method of wheel concentrations in determining the stresses in railway bridges, certain conditions of present practice should be considered carefully. The loading now in common use is that of wheel concentrations, and there seems to be no agreement among engineers as to the best substitute for this loading.

All the substitutes generally advocated involve the use of wheel concentrations in their computation. If the method of wheel concentrations is specified by purchasers, competition among bridge manufacturers compels the use of the actual wheel concentrations specified in computing stresses. Railway bridge engineers are in the habit of testing the strength of their existing structures by actual engines. They have moment diagrams calculated for the various types of engines in use on their system, and, naturally, prefer the method of wheel concentrations for calculating stresses. Any proposed change in the present method of specifying the load must meet with the approval of railway bridge engineers if it is to be adopted for general use.

Mr. Wing.

The principal objection urged against the use of the method of wheel concentrations is the greater labor involved in calculating stresses thereby.

This extra labor is so slight that its cost cannot be considered a sufficient reason for rejecting the method. Few engineers connected with bridge shops have admitted that the labor is great when a moment diagram has once been computed. Such a diagram is necessary for computing any of the substitute methods suggested.

The writer, therefore, suggests, as a solution of the question under discussion, that railway engineers, in writing specifications, include a moment diagram of the wheel concentrations as a part of the specification.

This will reduce materially the labor of the estimating department of every bridge manufactory, no matter what system of loading is finally adopted for computing stresses.

Such moment diagrams are part of the office equipment of every railway bridge engineering department, and including them in a specification involves no extra labor on the part of the bridge purchaser.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will hereafter be reproduced in the Volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

CHARLES EDWARD EMERY, M. Am. Soc. C. E.*

DIED JUNE 1ST, 1898.

Charles Edward Emery was born at Aurora, N. Y., March 29th, 1838, and was educated at the Canandaigua Academy. His first work was in manufactories and in the drawing offices of railroad shops in the vicinity of his home, after which he turned, for two years, to the study of law, with the intention of becoming a patent attorney. Upon the outbreak of the Civil War, Mr. Emery organized a company of volunteers, but on the announcement that the company was not needed he sought admission to the Navy. He was appointed Third Assistant Engineer in the Navy, July 11th, 1861, and was promoted to Second Assistant Engineer, December 18th, 1862. Mr. Emery's service included blockading duty with the Gulf Squadron and various engagements with the Confederate fleet below New Orleans, battles with forts at Pensacola, Fla.; with Forts Jackson and St. Philip, the capture of New Orleans and fights with the forts at Vicksburg and Port Hudson. Following his promotion, Mr. Emery was transferred to the *Nipsic*, the first of a new class of fast gunboats, stationed in Dahlgren's fleet, off Charleston, S. C. Some suggestions made by him, as to experimental steam apparatus, led to his selection, by the Chief Engineer of the Navy, in 1864, for scientific research in the United States Navy Steam Expansion Experiments at New York. Mr. Emery's labors for the Government did not, however, cease with his resignation from the Navy in 1868, for later he served as Consulting Engineer and Chairman of the Board of Examiners of the United States Revenue Marine and the United States Coast Survey, and, dating from April, 1872, as a member of the special commission advisory to the Secretary of the Treasury in reference to the Revenue Marine service.

Following his retirement from the Navy, Mr. Emery was engaged, in 1869, for a year or more, in making, for the Novelty Iron Works in New York City, a series of experiments upon stationary engines which it was proposed to manufacture. The records of these researches were published subsequently by Professor W. P. Trowbridge, and entitled "Condensing and Non-Condensing Engines." It had been

* Memoir prepared by the Secretary from papers on file, and from various sources.

arranged that Mr. Emery should become general superintendent of the works, but, owing to a change of plans, the establishment was closed, and Mr. Emery entered into business on his own account, as a consulting engineer.

Mr. Emery served as General Superintendent of the Fair of the American Institute in New York City in the autumn of 1869, and directed the scientific work of that institution in the testing of engines and boilers. In 1876 he was appointed a member of the international jury at the Centennial Exposition, on engines, pumps and mechanical appliances, and was associate to committees on other scientific apparatus. He inaugurated an extensive series of tests of engines and boilers, and prepared extended reports of this work, which were subsequently published. In 1893, Mr. Emery was appointed one of the judges of the World's Columbian Exposition, on dynamos and motors. He served in 1884 on the committee appointed by the American Society of Mechanical Engineers, to prepare a code for steam boiler trials, and, in 1895, was made chairman of a committee to revise the code previously reported. He was engaged in the final revision of this committee's report at the time of his death.

Mr. Emery's advisory services for the National Government began in 1869, with his appointment as Consulting Engineer to the Coast Survey and to the Revenue Marine. His engagement with these branches terminated when they passed, successively, into the control of the Navy Department, in 1879 and 1891. He fitted out nearly twenty of the revenue cutters, being in some cases responsible for the construction of the hulls, as well as the machinery. His experimental work in this service, in the construction of economical engines and in the scientific determination of engine-efficiency, attracted much attention. In 1874, he was able to place three different types of machinery in three hulls of the same size: first, a long-stroke, high-pressure, condensing engine; second, a short-stroke, low-pressure, condensing engine; and third, a fore-and-aft compound condensing engine. These vessels, as well as a subsequent one, in which the cylinder of a high-pressure condensing engine was jacketed, were tested by a joint board of engineers from the Navy and Treasury Departments, and the published results of this trial, together with an analysis of them by Mr. Emery, were widely circulated and used. Soon after this, New York University conferred upon him the degree of Doctor of Philosophy.

Possibly the most notable of Mr. Emery's engagements in private practice was the work of the New York Steam Company, which he took up in 1879. This involved the generation of steam in a central plant and its distribution for heating and power purposes. In its construction, the plant was the largest of the kind attempted, and included pioneer work in many details. The design and superintendence of the construction of this plant were in Mr. Emery's charge

until 1887, and, with its working, were described in a paper read before the Institution of Civil Engineers, in 1889. For that contribution, the Institution awarded him a Watt medal and a Telford premium.

Mr. Emery's practice as a consulting engineer and professional expert was extensive, and his services were in request in important legal cases. As consulting engineer for the City of Fall River, Mass., he was instrumental in bringing about a novel compromise of the difficulties between the city and the mill owners, resulting in an agreement whereby water was to be thereafter furnished to the city from the Watuppa ponds in consideration of the abatement of taxes on water power. He was a member of the commission on the purchase of the Long Island Water Supply Company's plant by the City of Brooklyn, N. Y.; was engaged in the matters of the water supplies of the cities of Newark, N. J., Lowell and Worcester, Mass.; the Holyoke, Mass., water-power tax cases, and had been consulting engineer for the Pneumatic Dynamite Gun Company. In 1887, he served as a member of the Board of Experts appointed to give an opinion as to the best method of increasing the number and size of trains on the New York and Brooklyn Bridge, by changes in the New York terminal, and at the time of his death he represented the Central Railroad of New Jersey in the matter of the Bound Brook floods.

Mr. Emery was elected a Member of the American Society of Civil Engineers May 6th, 1874. He contributed extensively to scientific literature and to the publications of the engineering societies in which he held membership. The papers by Mr. Emery published in the *Transactions* of this Society include: "Compound and Non-Compound Steam Engines, Steam Jackets, Etc.," December, 1874; "Connected-Arc Marine Boilers; Principles of their Construction," December, 1876; "Relative Quantities of Material in Bridges, of Different Kinds, of Various Heights," read at the Ninth Annual Convention, April 25th, 1877; "Steam Engine Economy," March, 1878; "Flexure and Transverse Resistance of Beams," June, 1879; "The Cost of Steam Power," June, 1883; "The Station B Chimney of the New York Steam Company," April, 1885; "A Novel Application of the Polar Planimeter," July, 1886; "Clamp for Pulling Sheet Piling," March, 1889; "Observations on the Forth Bridge," June, 1890, and "District Steam Systems," March, 1891. Mr. Emery was also a Member of the Institution of Civil Engineers, the American Society of Mechanical Engineers, the American Institute of Electrical Engineers, the American Institute of Mining Engineers, the American Association for the Advancement of Science and a Fellow of the Brooklyn Institute of Arts and Sciences.

SIR CASIMIR STANISLAUS GZOWSKI, M. Am. Soc. C. E.*

Died August 24th, 1898.

Casimir Stanislaus Gzowski was born at St. Petersburg, Russia, March 5th, 1813. He was the son of Stanislaus, Count Gzowski, who held a commission in the Imperial Guard, and was descended from an old Polish family, ennobled in the sixteenth century. At the age of 9 years the son was placed in the Military Engineering School at Kremnitz, and in 1830 he received a commission in the engineering branch of the Imperial service. He, in company with many other officers of his nationality, was active in the Polish insurrection of 1830-31, in consequence of which he was exiled, after several months spent in confinement in a military prison. He came to the United States and landed in New York City in the summer of 1833, a stranger to the language spoken, without friends and with but little money.

His first task was to acquire a knowledge of English. He had a command of Continental languages, and while pursuing his studies he obtained the means of livelihood by teaching French, German and Italian and by giving lessons in drawing and fencing. Removing to Pittsfield, Mass., he began the study of law with Mr. Parker Hall. He was admitted to citizenship in the United States and to the bar in Pennsylvania in 1837. He continued the practice of law for four years and then went to Toronto, with a view of securing a contract in connection with the widening of the Welland Canal.

Here he met some of the leading public men of the Dominion at that time, and Sir Charles Bagot, then at the head of the Canadian Government, formed a high opinion of his ability, and procured for Mr. Gzowski an appointment in the Public Works Department. This was the beginning of his long and distinguished career in the Dominion of Canada. For six years he was Superintending Engineer of Roads and Harbors in Western Ontario, and for three years engineer of the harbor works at Montreal and consulting engineer of the ship canal improvement between Montreal and Quebec. He became naturalized as a British subject, and in 1848 left the Department of Public Works to become Chief Engineer on the construction of the St. Lawrence and Atlantic Railway, now a part of the Grand Trunk system. He resigned this position in 1852 to enter into partnership with Sir Alexander Galt, Mr. Luther H. Halton and the Hon. D. L. Macpherson, for the construction of the Grand Trunk Railway from Toronto to Sarnia, from Port Huron to Detroit and from London to St. Mary's. In 1857 he established the Toronto Rolling Mills for the manufacture

*Prepared by the Secretary from information on file at the House of the Society.

of rails and other railway material. In 1870 he was appointed a member of a commission to inquire into the water communications of the Dominion, and in 1871 the firm of Gzowski & Macpherson constructed the International Bridge over the Niagara River. While in charge of this work, Mr. Gzowski was also called upon to report upon the enlargement of the Welland Canal and on the Baie Verte canal scheme. He was consulted frequently by the Dominion Government on matters relating to railways, canals and harbors.

Mr. Gzowski was chairman of the Niagara Falls Park Commission, one of the founders of the Canadian Society of Civil Engineers and its President for three consecutive years. He was deeply interested in military matters and in the promotion of international marksmanship contests. He rose to the rank of Colonel in the Canadian militia, and in 1879 was made an honorary aide-de-camp to the Queen. He held office in 1896, as Administrator of the Government of Ontario. In 1890, in recognition of valuable services rendered to the Dominion of Canada, he was created a Knight Commander of the Order of St. Michael and St. George.

Mr. Gzowski was elected a member of the American Society of Civil Engineers, December 2d, 1868, and served as one of its Directors during 1894.

FRANCIS RINECKER, M. Am. Soc. C. E.*

DIED APRIL 10TH, 1899.

Francis Rinecker, of Würzburg, Germany, was born December 16th, 1843. He studied mathematics and mechanical engineering under Professer F. Redtenbacher at the Polytechnic School at Carlsruhe, from 1858 to 1862, and civil engineering under Professor C. Culmann. at the Polytechnic School at Zurich, from 1862 to 1864. He began practical work in November, 1864, as assistant engineer of the Bavarian State Railways, C. Hohenner, chief engineer. He was engaged until November, 1867, in laying out and constructing part of the Munich and Ingolstadt Railway. In 1868 Mr. Rinecker traveled in Germany and France, and, coming to the United States, worked as an architectural draftsman from August of that year to April, 1869, in New York City, where he served with Charles Pfeifer, General Burger and J. Kellum. Turning to railway work in May, 1869, he was engaged successively as draftsman, assistant engineer and principal assistant of the construction department of the Pittsburg, Fort Wayne and Chicago Railway Company, and subsequently by the Pennsylvania Company, until June, 1873. Mr. Rinecker surveyed and constructed for the company first named the Erie and Pittsburg Railroad and the Ashtabula, Youngstown and Pittsburg Railroad, and conducted preliminary surveys for the Shenango Valley and Alliance Railroad, of which F. Slataper, M. Am. Soc. C. E., was chief engineer.

Returning to Europe, Mr. Rinecker served for two and a half years, from January, 1874, as Division Engineer of the Swiss Central Railway Company, for surveys and construction of the Liestal and Oensingen Railway and the Liestal and Waldenburg Railway (R. von Würthenau, Chief Engineer). From 1876 to 1883 he was engaged in literary pursuits with the especial view of introducing the rack on the St. Gothard and other mountain railways. He established in August, 1883, the firm of Rinecker, Abt & Co., at Würzburg, Germany, for the Abt system of mountain railways, and personally superintended the construction of the Lehesten & Oertelsbruch Railway and the narrow-gauge quarry tracks at Oertelsbruch and their extensions on the Abt combination system. From the establishment of this firm until his death, Mr. Rinecker had undertaken the construction of a number of Abt rack railways, including the Harzbahn in Germany, the Usui Toge Railway in Japan, the Transandine Railway, the Nilgiri Railway, the Mount Lyell Railway in Tasmania, the Mount Morgan Railway in Queensland and the Snowdon Railway.

* Memoir prepared from data furnished by Herr E. Wirsing, Würzburg, Germany.

In 1891–92 Mr. Rinecker traveled around the world, remaining in Cape Colony, New Zealand, Australia and Japan for several months. In these countries he made the acquaintance of many of the principal engineers, and studied the application of the Abt system to the conditions presented in each land. After his return he lived quietly at Würzburg, and died from influenza.

Mr. Rinecker was a member of the Institution of Civil Engineers, and was elected a Member of the American Society of Civil Engineers August 7th, 1872.

ROBERT DELOS ROWE, M. Am. Soc. C. E.*

Died March 14th, 1899.

Robert Delos Rowe was born at Sheridan, Ill., August 24th, 1863, and received a common-school education, with an advanced course in the graded school of that place, under the tutelage of Professor F. Y. Hamilton. He went into the field as rodman on the construction of the Atchison, Topeka and Santa Fé Railroad in Kansas, in 1878, and served as inspector of bridging in New Mexico in 1879–80, narrowly escaping the Indians during the Victoria troubles of 1882. Mr. Rowe returned to school in 1883–84, and after taking an engineering course at Valparaiso, Ind., under Professor Bogart, was advanced as Resident Engineer of the New Mexico Division of the Atchison, Topeka and Santa Fé Railroad, which position he held until 1892. Removing then to Chicago, he built the South Chicago City Railway and operated it for about two years. He then engaged in private practice in the engineering profession, became junior member of the firm of Rowe & Rowe of Chicago, planned the large timber preserving works at Somerville, Tex., and also similar works for the Santa Fé Pacific Railroad at Bellemont, Ariz.

Mr. Rowe married in 1885 Miss Allie Kyger, of Mulberry, Ind., who died one year later. He married again in 1897 Miss Milla Y. Campbell, of Chicago, who survives him. His death resulted from an attack of pneumonia. Mr. Rowe was a man of high acquirements and strict integrity, and had earned the confidence of his superiors and the love of his associates and employees.

Mr. Rowe was elected a Member of the American Society of Civil Engineers January 2d, 1890.

* Memoir prepared by Samuel M. Rowe, M. Am. Soc. C. E.

WILLIAM ULYSSES SCOTT, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 26TH, 1898.

William Ulysses Scott was born in Hoboken, N. J., January 18th, 1870. From his schoolboy days he showed remarkable mathematical abilities, and these brought him into prominence in his studies after his removal with his parents to Chicago, Ill., which occurred when he was twelve years old.

His first engagement was in March, 1888, with the Wheeling and Lake Erie Railroad, on the preliminary survey between Bowerston and Martins Ferry, Ohio. He spent nearly two years in the service of this road, rising to the position of assistant in the Toledo office, where he was engaged in making calculations for preliminary and location estimates. In the winter of 1889–90 he was employed with John Hanlon, of Canton, Ohio, on the preliminary survey for the Toledo and Walhonding Valley Railroad, and on the survey of the canal front at Massillon, Ohio. For two periods, in 1890 and 1891, he was employed as Assistant Engineer on the maintenance of way of the Second Division of the Illinois Central Railroad, and for three months preceding March, 1891, he was transitman on the location survey of the Fort Wayne, Terre Haute and Southwestern Railway.

In March, 1892, he again entered the employ of the Illinois Central Railroad Company as Assistant Engineer of the Louisiana Division. He had charge of a variety of engineering work, including the laying out of yards and company property, the construction of the Pacific Junction incline, the Poydras elevator, survey for the Stuyvesant dock terminal, and round houses at New Orleans, La., and Canton, Miss.

Mr. Scott had edited a book of logarithms, tables and short methods of computation, and was the author of numerous tables for frogs and switches, which, though never published, have been found of service by many engineers. He was also expert in laying out and personally supervising intricate track work.

He was modest and unassuming in manner, highly esteemed by the officials of the companies he served, and popular with those persons with whom his work brought him in contact.

Mr. Scott was elected an Associate Member of the American Society of Civil Engineers, February 3d, 1897.

* Memoir prepared by the Secretary from papers on file, and from information furnished by Hartwell P. Farrar, B. M. Harrod and W. W. Hayden, Members, Am. Soc. C. E.

HISTORICAL SKETCH

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS,

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Two thousand copies were printed; 300 were bound in full morocco, of which 216 have been sold at $10 per copy, the resulting net profit being $943.06. Seventeen hundred copies, which have been paid for, are still on hand, and the Board of Direction was requested to consider the propriety of offering to the membership these copies bound in a less expensive style and at a reduced price, the net proceeds to be applied to the building fund.

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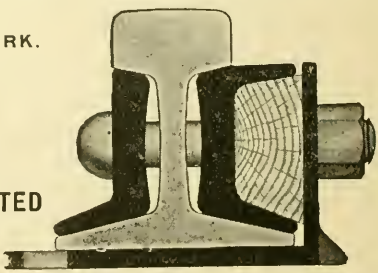

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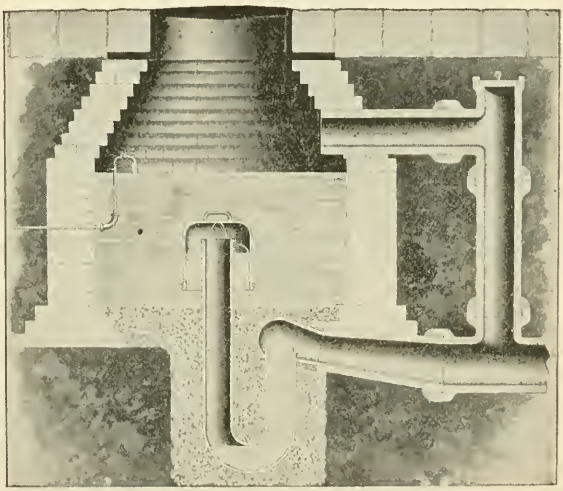

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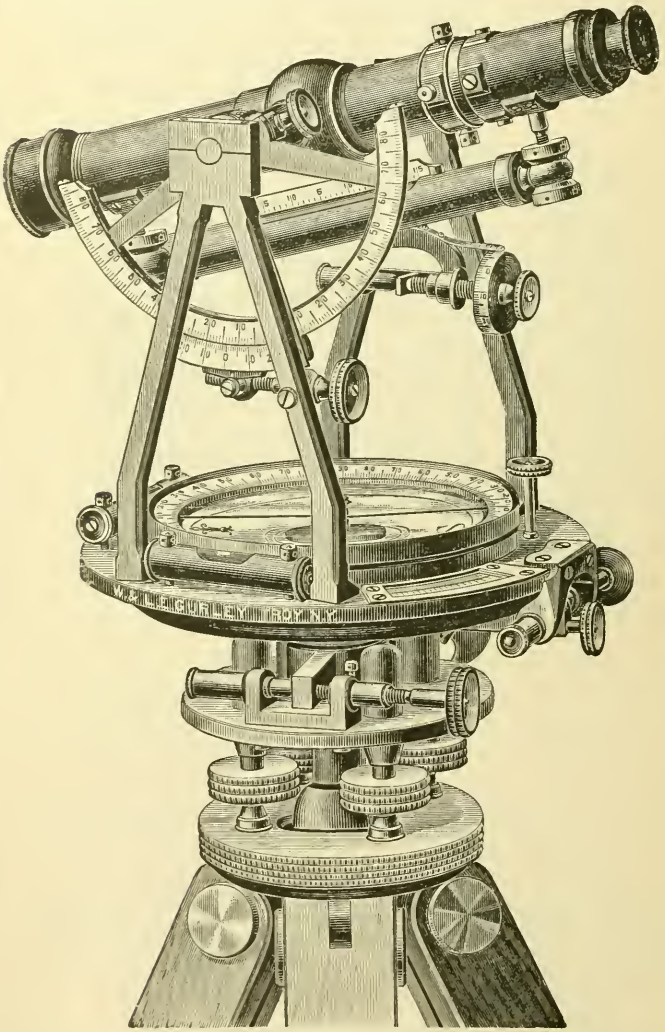

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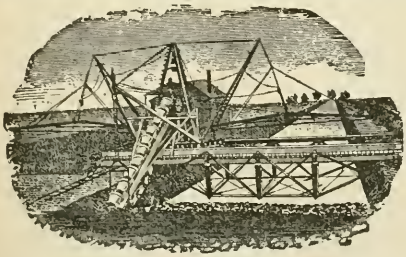

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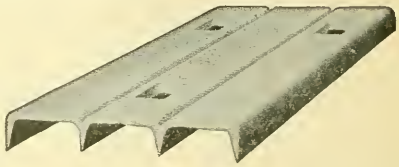

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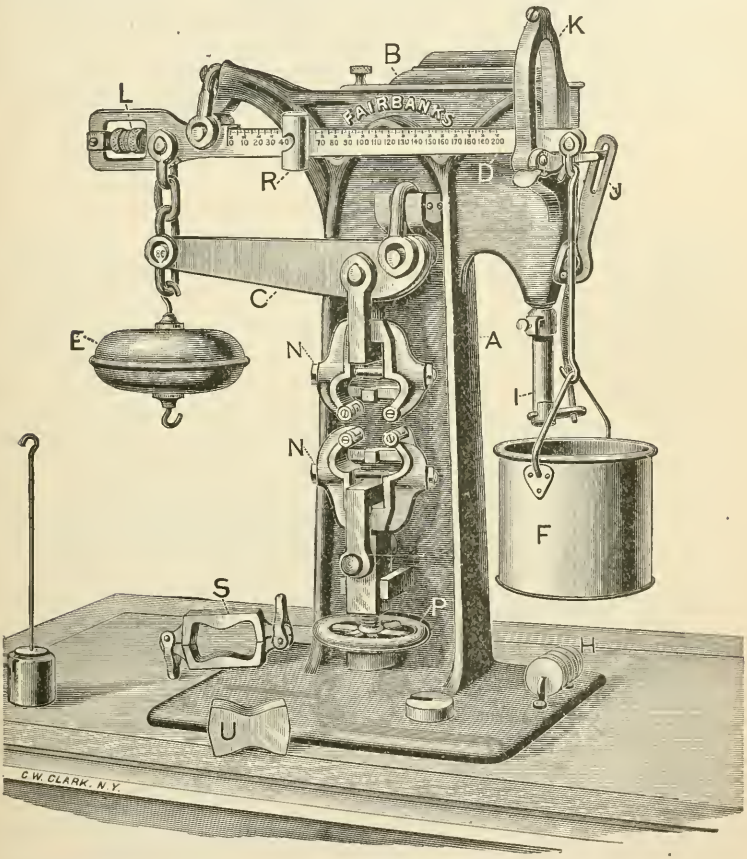

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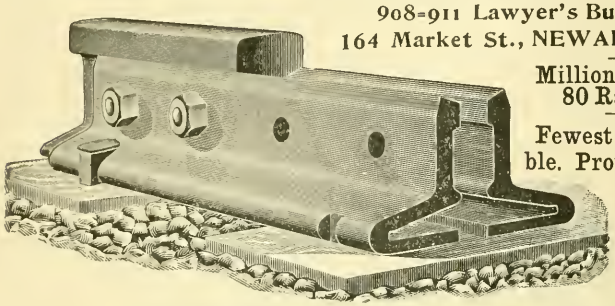

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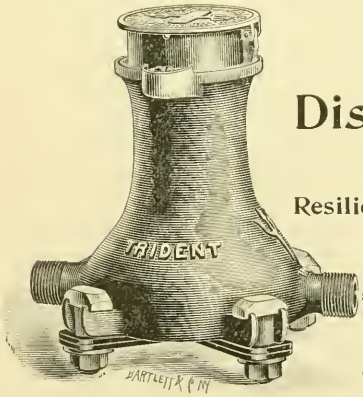

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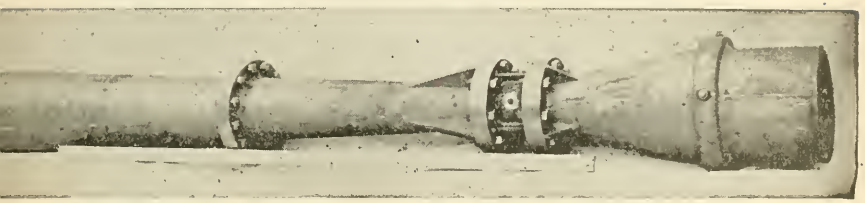

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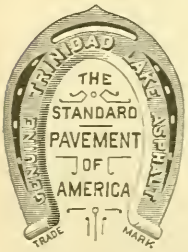

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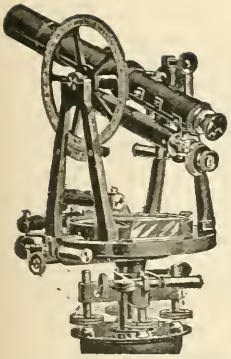

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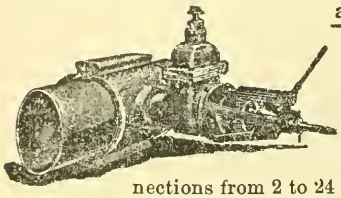

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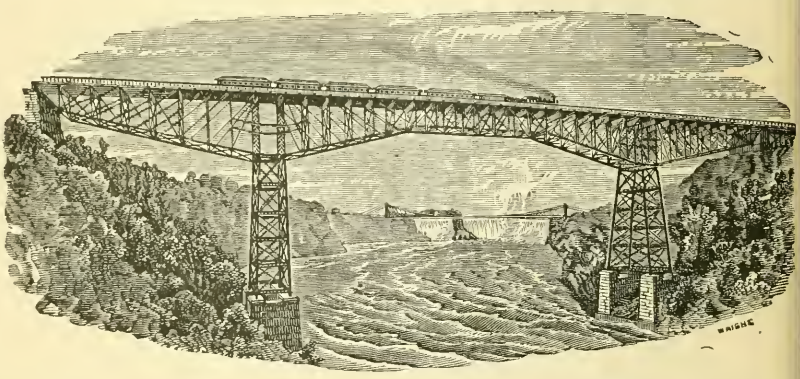

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Vol. XXV. NOVEMBER, 1899. No. 9.

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publications

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The prices of publications are as follows: *Proceedings*, $6 per annum; *Transactions*, $10 per annum. Postage will be added when *Proceedings* are sent to foreign countries.

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ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Noble, George S. Webster, O. M. Carter, W. B. W. Howe, Louis C. Sabin, H. W. York.

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Vol. XXV. NOVEMBER, 1899. No. 9.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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MINUTES OF MEETINGS.

OF THE SOCIETY.

November 1st, 1899.—The meeting was called to order at 8.40 P. M., Joseph M. Knap in the chair; Charles Warren Hunt, Secretary; and present, also, 65 members and 10 visitors.

The minutes of the meetings of October 4th and 18th were approved as printed in *Proceedings* for October, 1899.

A paper entitled "Test of a Mechanical Filter," by Edmund B. Weston, M. Am. Soc. C. E., was presented by the Secretary, who also presented a written discussion on the subject by Mr. Gardner S. Williams.

The paper was discussed orally by Messrs. W. B. Fuller, E. Sherman Gould, A. P. Folwell and C. G. Currier.

Ballots were canvassed, and the following candidates declared elected.

As Members.

Frank Lynton Chase, Louisville, Ky.
George Richardson Harlow, Wilkinsburg, Pa.
John Francis Healy, Red Lodge, Mont.

As Associate Members.

Edmund Ignatius Bowen, Hornellsville, N. Y.
William Saxon Charlesworth, Auckland, New Zealand.
James William Martin, Boston, Mass.
Soloman Mark Swaab, Philadelphia, Pa.
Frederick Kelly Wing, Buffalo, N. Y.

Announcement was made that the following candidates were elected by the Board of Direction, October 31st, 1899:

As Juniors.

Willard Franklin Graves, South McAlester, Ind. T.
Henry Alexander Harris, Mapimi, Durango, Mex.
Louis Bernard Spencer, Salt Lake City, Utah.
Charles Eliphalet Trout, Brooklyn, N. Y.

The Secretary announced that the Annual Convention of 1900 would be held in London, England, but that the date had not yet been fixed (see p. 226).

Adjourned.

November 15th, 1899.—The meeting was called to order at 8.30 P. M., Director H. S. Haines in the chair; Charles Warren Hunt, Secretary, and present, also, 73 members and 15 visitors.

A paper by Frank H. Cilley, S. B., entitled "The Exact Design of Statically Indeterminate Frameworks: An Exposition of Its Possibility, but Futility," was presented by the author and discussed by Mr. Henry Goldmark.

Mr. William S. Dalrymple addressed the Society on the subject "The Triangulation of Greater New York." The address was illustrated with stereopticon views.

The Secretary announced the death of Clarence Allan Carpenter, elected Member May 2d, 1888; died November 9th, 1899.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

October 31st, 1899.—Vice-President North in the chair; Charles Warren Hunt, Secretary, and present also Messrs. Bensel, Haines, Manley, Morison, Owen, Ricketts, Thomson, Turner and Whinery.

The recommendation of the Finance Committee, that a payment of $10 000 be made on January 15th, 1900, on the loan now standing against the Society House, was adopted.

The recommendation of the Finance Committee, that all entrance fees received by the Society shall, in future, beginning with the year 1900, be set aside as a fund to extinguish the debt, and that a sum not less than the amount of such entrance fees be paid yearly on this debt until it shall be extinguished, was adopted.

On the recommendation of the Library Committee the employment of temporary additional assistants in cataloguing, in order to hasten the completion of the index, was authorized.

A report from the Special Committee on Time and Place for holding the next Annual Convention was received, which presented the following facts:

That 671 answers to Convention Circular, No. 2, had been received; that of these 408 favored the holding of a Convention in London, England; that 217 were in favor of the alternate proposition of sending a delegation; that 178 votes were received against a London Convention, and 225 against the idea of holding a Convention in America and sending a delegation to Europe. The balance of the votes were either in favor of both propositions, against both propositions, indifferent as to the matter, or in favor of some entirely different proposition.

It was decided that the Thirty-Second Annual Convention be held in London, England, the time to be determined upon further investigation.

Applications were considered, and other routine business transacted.

Four candidates for Junior were elected.*

Adjourned.

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* See page 224.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society is open every day, except Sunday, from 9 A. M. to 10 P. M.

ANNUAL CONVENTION OF 1900.

At a meeting of the Board of Direction, October 31st, 1899, the results of the last circular of inquiry, which was sent out by a Special Committee, as well as the results of two previous circulars, were discussed, and as the results of each of these circulars indicated that a majority of such members as cared to vote, favored a Convention to be held abroad, the Board decided that the Convention of 1900 be held in London, England.

No definite date has yet been decided upon for the holding of this Convention, and the fixing of this date will be deferred until more information can be secured, as to the most available steamers, and the general convenience of the members who desire to go.

With a view of securing some information on this last matter members are requested to write the Secretary, giving their views as to what general date would be convenient to them, and the reasons governing them in that opinion.

JUNIOR MEETINGS.

The meetings of the Juniors of the Society were inaugurated for the season of 1899-1900, by a meeting on October 25th, 1899, at which the subject "The Relation of Design to Durability" was considered, the discussion being opened by J. B. Hurtig, Jun. Am. Soc. C. E.

MEETINGS.

Wednesday, December 6th, 1899, at 8.30 P. M., a regular business meeting will be held. Ballots for membership will be canvassed, and a paper by James A. Seddon, M. Am. Soc. C. E., entitled, "River Hydraulics," will be presented. This paper was printed in the October number of *Proceedings*.

At this meeting, A. C. Savage, M. Am. Soc. C. E., will address the Society on "The Present Condition and Prospective Development of Wire-Rope Transmission." This address will be illustrated with lantern-slides.

Wednesday, December 20th, 1899, at 8.30 P. M., a regular meeting will be held, at which a paper by A. H. Sabin, Assoc. M. Am. Soc. C. E., entitled, "Experiments on the Protection of Steel and Aluminum Exposed to Water," will be presented. This paper was printed in the October number of *Proceedings.*

Wednesday, January 3d, 1900, at 8.30 P. M., a regular business meeting will be held. Ballots for membership will be canvassed, and a paper by Allen Hazen, Assoc. M. Am. Soc. C. E., entitled "The Albany Water Filtration Plant," will be presented. This paper is printed in this number of *Proceedings.*

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(October 11th to November 7th, 1899.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Eng. Clnb of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Eng., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Can.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Institute, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 30c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 35c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City, 35c.
(18) *Railway and Engineering Review*, Chicago, Ill.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England.
(22) *Iron and Coal Trades Review*, London, England.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gaslight Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Electrical Engineer*, New York City, 10c.
(28) *Industries and Iron*, London, England.
(29) *Journal*, Society of Arts, London, England.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des Ecôle Spéciales de Gand*, Brussels, Belgium.
(32) *Memoirs et Compt Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mecanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Transactions*, Am. Inst. Elec. Eng., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 75c.
(45) *Mines and Minerals*, Scranton, Pa. 20c.
(46) *Scientific American*, New York City, 10c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift des Vereines Deutscher Ingenieure*, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Duesseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Ger many.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereines*, Vienna, Austria.
(54) *Den Tekniske Forenings Tidsskrift*, Copenhagen, Denmark.
(55) *Ingeniören*, Copenhagen, Denmark.
(56) *Teknisk Tidskrift*. Stockholm, Sweden.
(57) *Teknisk Ugeblad*, Christiania, Norway.
(58) *Proceedings*, Eng. Soc. W. Pa. 410 Penn Ave., Pittsburg, Pa. 50c.
(59) *Transactions*, Mining Institute of Scotland, London and Newcastle-upon-Tyne.
(60) *Bridges and Framed Structures*, 358 Dearborn St., Chicago, Ill., 30c.
(61) *Proceedings*, Western Railway Club. 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.

LIST OF ARTICLES.

Bridge.

Concrete Protection Walls for Bridge Supports. (18) Oct. 14.
The Scherzer Rolling-Lift Bridge, (47) Oct. 7.
The Development of the Arch Principle. (12) Oct. 6.
The Eighteenth Street Bridge, Philadelphia. (14) Oct. 14.
Suspension Bridges with Stiffening Girders. Max am Ende. (63) Part III, 1899.
Oblique Arches on Curves. Arthur Hatsell Owles. (63) Part III, 1899.
A Concrete and Iron Bridge at Oconomowoc, Wis. (13) Oct. 19.
Manufacture of the Cast-Steel Voussoirs for the Alexander III Bridge, Paris. (13) Oct. 19.
A Long Double-Track Plate Girder Span. (14) Oct. 21.
The Life of Timber Employed in Railway Bridges in the United States. (18) Oct. 21; (13) Oct. 26; (15) Oct. 27.
The Maiden Lane Bridge, Albany, N. Y. (14) Oct. 28.
The Atbara Bridge. (14) Nov. 4.
Concrete Piers, Sangamon River Bridge. (14) Nov. 4.
New Form of Plate-Girder Swing Bridge, C. M. & St. P. Ry. (18) Oct. 28.
Le Pont Roulant de Montage du Pont Alexandre III à Paris. (35) Oct., 1899.
Les Travaux d'Amélioration du Pont de Huy. E. Marote. (31) Troisième livraison, 1899.
Execution d'un Pont Biais en Ciment Armé d'après le Système Hennebique. E. Haerens. (31) Troisième livraison, 1899.

Electrical.

The Determination of the Wave Form of Alternating Currents without a Contact Maker. Harris J. Ryan. (42) Aug. and Sept., 1899.
The Protection of Secondary Circuits from Fire Risk. Cary T. Hutchinson. (42) Aug. and Sept., 1899.
Measurement of the Internal Resistance of Accumulators. E. C. Rimington. (26) Oct. 20.
Apparatus for Obtaining High Frequencies and Pressures. Elihu Thomson. (27) Oct. 14.
A Multiple Lightning Flash. A. E. Kennelly. (27) Oct. 28.
The Mechanism of the Wehnelt Interruptor. (26) Oct. 20.
Notes on Electrical Specifications. Ernest Kilburn Scott. (26) Oct. 6.
Interior Alternating Current Wiring. R. W. Lohmann. (27) Oct. 21.
The Electric Driving of Printing Machinery. (26) Oct. 6.
Motor Speed Regulation. Alton D. Adams. (42) Aug. and Sept. 1899.
Electricity for the Auxiliary Machinery on Warships. Lieut. J. K. Robison. (9) Nov., 1899.
German Electric Automobiles. (27) Nov. 4.
Protection of Interior Wiring from High-Potential Currents. Warren B. Reed and Lyman C. Reed. (27) Oct. 28.
Plea for a Working Standard of Light. S. Everett Doane. (42) Aug. and Sept., 1899.
The Photometry of Incandescent Lamps. Prof. Arthur J. Rowland. (3) Nov., 1899.
Series Alternating Current Arc Lighting from Constant Current Transformers. (27) Nov. 4.
Incandescent Lamp and Transformer Construction in America. Henry E. P. Cottrell. (12) Serial beginning Oct. 13, ending Oct. 20.
Enclosed Arc Lamp Statistics. L. B. Marks. (27) Nov. 4.
Woburn, Mass., New Central Station. L. R. Wallis. (27) Oct. 28.
The Hull (Que.) Electric Company and the Deschene Electric Company. E. M. Archibald. (27) Oct. 14.
West Ham, England, Central Station. (27) Nov. 4.
Carlisle Corporation Electricity Works. (26) Oct. 6.
Llandudno Municipal Electricity Works. (26) Oct. 13.
The Commercial Transformation of Energy. Daniel W. Mead, M. Am. Soc. C. E. (28) Serial beginning Sept. 15, ending Oct. 13.
Means of Attaining Safety in Electrical Distribution. W. L. R. Emmet. (47) Oct. 14.
Electric Power Distribution and the Small Consumer. Dr. Louis Bell. (9) Nov., 1899.
An 83-Mile Electric Power Transmission Plant. James A. Lighthipe. (10) Nov., 1899.
The Snoqualmie Falls Electric Power Plant, (11) Oct. 6; (27) Oct. 21.
Mt. Whitney, Cal., Transmission Plant. Donald H. Fry. (27) Oct. 28.
The Relation Between Electrical Machines and Prime Movers. R. M. Friese. (48) Sept. 30.
Electrical Power in Rolling Mills and Foundries. O. Lasche. (50) Oct. 1.
La Commande Directe des Machines-Outils, par des Moteurs à courants Polyphases. (36) Oct. 10.
Distribution d'Énergie Électrique de Nuremberg. Henry Martin. (33) Sept. 30.

Marine.

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The Development of German Shipbuilding—Yards for the Building of River Boats. Rudolph Haack. (9) Nov., 1899.
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The Machinery of the Graving Dock at Govan. George H. Baxter. (47) Serial beginning Sept. 16, ending Oct. 7.
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Mechanical.

The Niclausse Water-Tube Boiler. Mark Robinson. (47) Serial beginning Oct. 7, ending Oct. 14.
Recent Practice in Steam Boilers in Great Britain. W. D. Wansbrough. (10) Nov., 1899.
Water-Tube Boilers for Marine Engines. James Tayler Milton. (63) Part III.
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Smokeless Combustion of Soft Coal. William Kent. (8) Oct., 1899.
The Safety of Petroleum Fuel—Its Use for Locomotives and for Stationary Boilers. James Holden. (10) Nov., 1899.
Crude Petroleum and its Products as Fuel. H. Tweddle. (16) Serial beginning Oct. 14, ending Oct. 28.
A New Open-Hearth Steel-Making Process. Alexander Sattman. (22) Oct., 20.
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Some Forms of Magnetic Separators and their Application to Different Ores. H. C. McNeill. (11) Serial beginning Oct. 13, ending Oct. 20.
Some Recent Applications of Electro-Metallurgy to Mechanical Engineering. (22) Oct. 6.
Holding Power of Wood Screws. W. M. Macphail and T. T. Irving. (11) Oct. 6.
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Evolution of Mining and Ore Treatment in Colorado. Thomas Tonge. (9) Nov. 1899.
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The Street Railway System of Hanover, Germany. Woolsey McA. Johnson. (27) Oct. 14.
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The Salient Features of the Chief Engineer's Annual Report of the Drainage Canal of the Sanitary District of Chicago for 1898. Isham Randolph. (4) Aug., 1899.
Sewage. Sir W. H. Preece. (28) Oct. 13.
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Pig Iron Fractures and their Value in Foundry Practice. James Williard Miller. (62) Oct. 19; (13) Nov. 2.
The Relation Between the Structure of Steel and Its Thermal and Mechanical Treatment. Albert Sauveur. (47) Serial beginning Oct. 21, ending Oct. 28.
Testing of Chimney Masonry. (55) Sept. 16.
Comparisons of Similar Structures and Machines. Prof. Archibald Barr. (11) Serial beginning Oct. 13, ending Oct. 27.
Some Notes on Fireproofing and Preserving Timber. Sherard Cowper-Cowles. (28) Oct. 20.
Experiments on Earth Pressure Against Retaining Walls. A. A. Steel. (13) Oct. 19.
Masonry Dams—Principles of Design and Construction. Robert S. Ball, Jr. (10) Nov., 1899.
A Simple Method of Bonding the Brickwork of a Square Arch with Oblique Faces. Henry Ashman Reed. (63) Part III. 1899.
The Foundations of the Manchester Ship-Canal Grain-Elevator. Gerald Gascoigne Lynde. (63) Part III. 1899.
Elasticity of Portland Cement. W. L. Brown. (63) Part III. 1899; (14) Oct. 28.
Specifications for Waterproofing Tunnel Masonry. (14) Oct. 28.
Rapid Pneumatic Foundation Work. (14) Oct. 28.
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The Water-Works of the Madras Presidency. John Alfred Jones. (63) Part III. 1899.
Coatings for Cast-Iron Water Pipes. Thomas H. Wiggin. (47) Oct. 7.
Laying a 6-In. Water Main Across Havana Harbor. Howard Egleston. (13) Oct. 12.
The Reduction of Water-Waste. (14) Oct. 14.
Water Rates. (14) Oct. 14.
Lining a Reservoir near Whitby. George Bransby Williams. (63) Part III. 1899.
Covered Reservoirs. Frank L. Fuller. (1) Sept., 1899.
Sedimentation Basins for Water-Works. Wynkoop Kiersted, M. Am. Soc. C. E. (14) Oct. 28.
Elements of Design Favorable to Speed Regulation in Plants Driven by Water Power. Allan V. Garratt. (42) Aug. and Sept., 1899.
Les Travaux de Dérivation des Sources du Bocq. Ed. Walin. (30) Oct., 1899; (31) Troisième livraison. 1899.
Barrages-Réservoirs en Remblai Rochaux avec âme Métallique. A. Dumas. (33) Oct. 21.

Waterways.

Notes on Chicago City Datum. John W. Alvord, M. Am. Soc. C. E. (4) Aug., 1899.
The Marquette Breakwater and Duluth Ship Canal Piers. (14) Oct. 28.
The Dymchurch Wall and Reclamation of Romney Marsh. Edward Case. (12) Oct. 20.
Locks and Lock Gates for Ship Canals. Henry Goldmark, M. Am. Soc. C. E. (1) Sept., 1899.
Sound-Producing Buoy. (28) Oct. 27.
Improvements in Dioptric Apparatus for Light-Houses. William Tregarthen Douglass and John Archibald Purves. (63) Part III. 1899.
Test of the Kretz Dredge at Strassburg. (51) Serial beginning Oct. 4, ending Oct. 7.
Le Canal de Dortmund à l'Ems, et l'Ascenseur pour Bateaux d'Henrichenburg. Georges Henry. (33) Oct. 7.
Les Travaux à l'Air Comprimé au Port d'Ostende. M. A. Cadola. (30) Oct., 1899.
L'Exhaussement du Barrage de la Fonderie de Canons, à Liége. G. Herman. (30) Oct., 1899.
Usine Hydraulique de Bourg-et-Comin pour l'Alimentation du Canal de l'Oise à l'Aisne. (34) Oct., 1899.

NEW BOOKS OF THE MONTH.

Unless otherwise specified, books in this list have been donated to the Library by the Publisher.

MONOPOLIES AND THE PEOPLE.

By Charles Whiting Baker. Edition 3, revised and enlarged. Cloth 9 x 6 inches, 368 pp. New York, G. P. Putnam's Sons, 1899. $1.50.

The older portion of this work has been reprinted in its original form. New material has been added, presenting a survey of the growth of monopolies in each of the several departments of industry during the past decade. This review has been confined, with few exceptions, to the United States. The author is of the opinion that the death of competition, in a great proportion of industries, is inevitable, and that Government regulation is likewise inevitable.

THE ACTION OF MATERIALS UNDER STRESS;

Or, Structural Mechanics; comprising the Strength and Resistance of Materials and Elements of Structural Design; with Examples and Problems. By Charles E. Greene, M. Am. Soc. C. E., Professor of Civil Engineering, University of Michigan. Cloth, 9 x 6 ins.. 266 pp., 3 plates. Ann Arbor, Mich., 1897. Printed for the Author at the Inland Press. Donated by the Author.

This work is intended by the Author to be serviceable after class-room work is concluded, and is arranged for reference, with that end in view. The treatment of the subject is largely graphical. All the constructions are simple. The notation, throughout, is practically uniform, and is that used by several standard authors. The Contents are: Action of a Piece under Direct Force; Materials; Beams; Torsion; Moments of Inertia; Flexure and Deflection of Simple Beams; Restrained Beams; Continuous Beams; Pieces under Tension; Compression Pieces; Safe Working Stresses; Internal Stress, Change of Form; Rivets; Pins; Envelopes; Plate Girder; Earth Pressure; Details in Wood and Iron. There is an index of three pages.

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ADDITIONS.

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Eng., N. P. Ry., Freeburg, Minn.		Oct. 4, 1899
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Prof., Civ. Eng., Washington Univ., St. Louis, Mo	Assoc. M. M.	Oct. 3, 1894 Oct. 4, 1899

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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THE ALBANY WATER FILTRATION PLANT.

By ALLEN HAZEN, Assoc. M. Am. Soc. C. E.

TO BE PRESENTED JANUARY 3D, 1900.

HISTORICAL.

Albany, N. Y., was originally supplied with water by gravity from certain reservoirs on small streams west and north of the city. In time, with increasing consumption, the supply obtained from these sources became inadequate, and an additional supply from the Hudson River was introduced. The water was obtained from the river through a tunnel under the Erie Basin, and a pumping station was erected in Quackenbush Street to pump it to reservoirs, one of which served also as the distributing point for one of the gravity supplies. The intake, which was used first in 1873, drew water from the river opposite the heart of the city. In recent years, the amount of water drawn from this source has greatly exceeded that obtained from the gravity sources.

Some of the city sewers enter the river above the intake, but most of them are below it. In times of flood, the water thus obtained was polluted by the sewage of only a few of the city sewers. At low-water stages, however, owing to the tidal currents, the water contained much sewage, which was carried up stream to the intake, and

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in the volumes of *Transactions*.

the sewage of the city was thus present, in very considerable amount, in its own water supply.

In addition to the local sources of pollution, the river received the sewage of Troy and the surrounding cities, 7 or 8 miles above, and the sewage of Schenectady, Utica, Rome, and many other places farther away.

Under these conditions the typhoid fever death rate in Albany was excessive. Professor W. P. Mason, of Troy, made a report upon the quality of the water to the Water Board in 1885, in which he stated in unmistakable terms that the water as then used was a source of disease, and should be abandoned at the earliest practicable date.

Following this, an attempt was made to secure a ground-water supply, but without results. Studies were then made for gravity sources of supply, mentioned in the reports of the Board of Water Commissioners for the years 1891 to 1893.

Failing to secure the necessary legislation to introduce a gravity supply, the Board, in 1896, investigated methods of purifying the present supply. The matter was studied by the Board and by its Superintendent, George I. Bailey, M. Am. Soc. C. E., and the author was engaged, in January, 1897, to examine the studies which had been made, and to report upon the projects presented. This report, presented to the Board in February, 1897, recommended the general scheme previously outlined by the Superintendent, namely, to abandon the present intake, and to establish a new one at a point about two miles farther up the river, at a point above all the local sources of pollution, and to pump the water by low-lift pumps to a settling basin, from which it would flow to sand filters, and thence through a pure-water conduit to the present pumping station in Quackenbush Street.

Among other considerations, taken into account in determining upon this site, was the fact that the present high-service reservoir is not at a sufficient elevation to supply properly the highest parts of the city, although it is on the highest ground in its immediate neighborhood. It will be necessary ultimately to construct a new high-service reservoir, and the most available location for it is upon land about $1\frac{1}{2}$ miles northeast of the present Prospect Hill Reservoir, where there is an excellent site directly back of that recommended for the filters; so that when it becomes necessary to construct the reservoir,

the most advantageous location for a pumping station to supply it will be close to the filters.

The report was accepted, the recommendations adopted, and the necessary funds were provided; and, in July, 1897, the preparation of plans was begun.

As the pure-water conduit was to be placed in the Erie Canal, and as the work of constructing it necessarily had to be done during the season of closed navigation, plans for this part of the work were prepared first. The contract was let to Messrs. Hilt, Johnson, Fitzgerald & Mulderry in November, 1897, and that part of the work under the canal was completed before navigation opened, May 1st, 1898.

Plans for the filters and sedimentation basin were completed in January, 1898, and the contract was awarded in February to Messrs. T. Henry Dumary and the Wilson and Baillie Manufacturing Company, who commenced work under their contract in April; but, owing to various reasons connected with the installation of a very elaborate contractors' plant, work was not pushed actively by them until August, 1898. Contracts for the pumping machinery, and for the pumping station and intake, were let to the Prindle Pump Company and to Stone & Thurston, respectively, in June and August, 1898, and the work was carried out during the fall and winter. Gates and all special valves were furnished by the Eddy Valve Company. The work was sufficiently advanced so that a part of the plant was put in operation on July 27th, 1899. The old intake was closed on September 6th, 1899, since which time no unfiltered river water has been pumped to the city.

Source of Supply.

The Hudson River, at the point of intake, has a drainage area of 8 240 square miles. Of this, 4 570 square miles are tributary to the Hudson above Troy, 3 502 are tributary to the Mohawk, and 168 are tributary to the Hudson below the Mohawk.

The average annual flow of the streams probably amounts to at least 1 000 000 galls. per square mile per day, or over 8 000 000 000 galls. per day. The minimum flow is only a small fraction of this amount. No gaugings of the river at this point are available. George W. Rafter,* M. Am. Soc. C. E., has made a study of the flow of the Upper Hudson, and estimates the minimum flow at Glens Falls at 0.24

* Report of the State Engineer and Surveyor for 1895, p. 119.

cu. ft. per second per square mile of tributary drainage area. Assuming that this figure applies to the whole of the Hudson above Troy, and taking a somewhat lower figure, namely, 0.15 cu. ft. per second per square mile, for the discharge of the Mohawk and of the Hudson

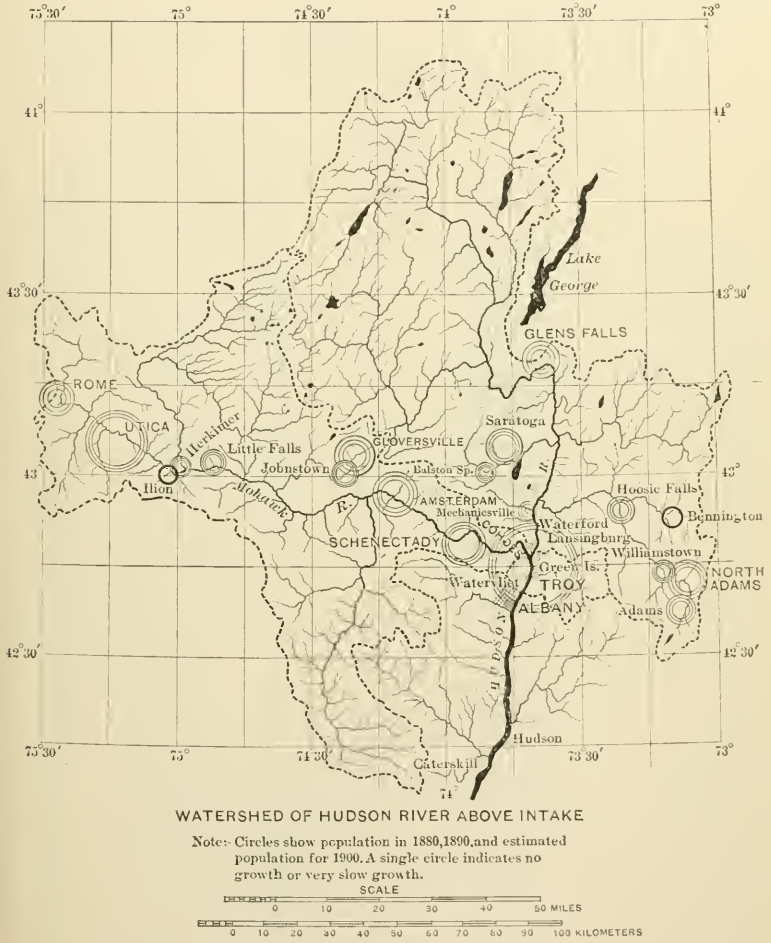


FIG. 1.

below Troy, we arrive at a minimum flow of the Hudson at Albany of 1 657 cu. ft. per second, or 1 060 000 000 galls. per 24 hours, being, in round numbers, 100 times the average amount of water now taken from the river for water-works purposes, and at least 50 times the maximum.

Pollution of the Raw Water.—Table No. 1 gives the names of the cities and larger towns upon the river above the intake, with estimated populations and distances. The largest of these places are also shown upon the map of the water-shed, Fig. 1.

TABLE No. 1.—Cities, Towns and Villages on the Water-Shed of the Hudson River above Albany, with Populations of 1 000 and Over.

Place.	County.	Approximate distance above intake. Miles.	Population in:		
			1880.	1890.	1900. (Estimated.)
Troy	Rensselaer	4	56 747	60 956	65 470
Watervliet	Albany	4	8 820	12 967	19 040
Green Island	Rensselaer	5	4 160	4 463	4 788
Waterford	Saratoga	5	(1 822)	1 822	(1 822)
Cohoes	Albany	8	19 416	22 509	26 450
Lansingburg	Rensselaer	8	7 432	10 550	14 980
Mechanicsville	Saratoga	19	1 265	2 679	5 358
Schaghticoke	Rensselaer	27	(1 258)	1 258	(1 258)
Schenectady	Schenectady	28	13 655	19 002	26 450
Schuylerville	Saratoga	32	1 617	1 387	1 190
Greenwich	Washington	39	1 251	1 663	2 247
Fort Edward	Washington	43	4 680	4 424	4 182
Hoosic Falls	Rensselaer	44	4 530	7 014	10 860
Amsterdam	Montgomery	44	9 466	17 336	31 730
Cambridge	Washington	45	1 482	1 598	1 723
Sandy Hill	Washington	46	2 487	2 895	3 371
Glens Falls	Warren	49	4 900	9 509	18 450
South Glens Falls	Saratoga	49	1 083	1 606	2 387
Saratoga Springs	Saratoga	51	8 421	11 975	17 010
Ballston Springs	Saratoga	51	3 011	3 527	4 131
Fultonville	Montgomery	54	881	1 122	1 429
Fonda	Montgomery	54	944	1 190	1 500
Johnstown	Fulton	56	5 013	7 768	12 040
Bennington, Vt.	Bennington	56	(3 971)	3 971	(3 971)
Gloversville	Fulton	58	7 133	13 864	26 930
Canajoharie	Montgomery	63	2 013	2 089	2 168
Williamstown, Mass.	Berkshire	63	3 394	4 221	5 250
Corinth	Saratoga	64	510	1 222	2 444
North Adams, Mass.	Berkshire	68	10 191	16 074	25 340
Luzerne	Warren	68	468	868	1 610
Schoharie	Schoharie	68	1 188	1 028	889
Fort Plain	Montgomery	69	2 443	2 864	3 358
Middleburg	Schoharie	73	1 123	1 139	1 155
St. Johnsville	Montgomery	74	1 072	1 263	1 488
Cobleskill	Schoharie	74	1 222	1 822	2 717
Adams, Mass.	Berkshire	75	5 591	9 213	15 181
Warrensburg	Warren	82	748	893	1 073
Little Falls	Herkimer	82	6 910	8 783	11 160
Herkimer	Herkimer	90	(1 353)	(1 353)	(1 353)
Mohawk	Herkimer	91	1 441	1 806	2 264
Ilion	Herkimer	93	3 711	4 057	4 436
Frankfort	Herkimer	95	1 085	2 291	4 582
Utica	Oneida	104	33 914	44 007	57 090
Whitesboro	Oneida	107	1 370	1 663	2 018
New York Mills	Oneida	108	(1 195)	1 195	(1 195)
New Hartford Mills	Oneida		710	912	1 171
Oriskany	Oneida	111	597	860	1 239
Clinton	Oneida	121	1 236	1 269	1 303
Rome	Oneida	121	12 194	14 991	18 430
Waterville	Oneida	127	(1 734)	1 734	(1 734)
Total, not including rural population			272 838	354 672	479 415
Per square mile			33	43	59

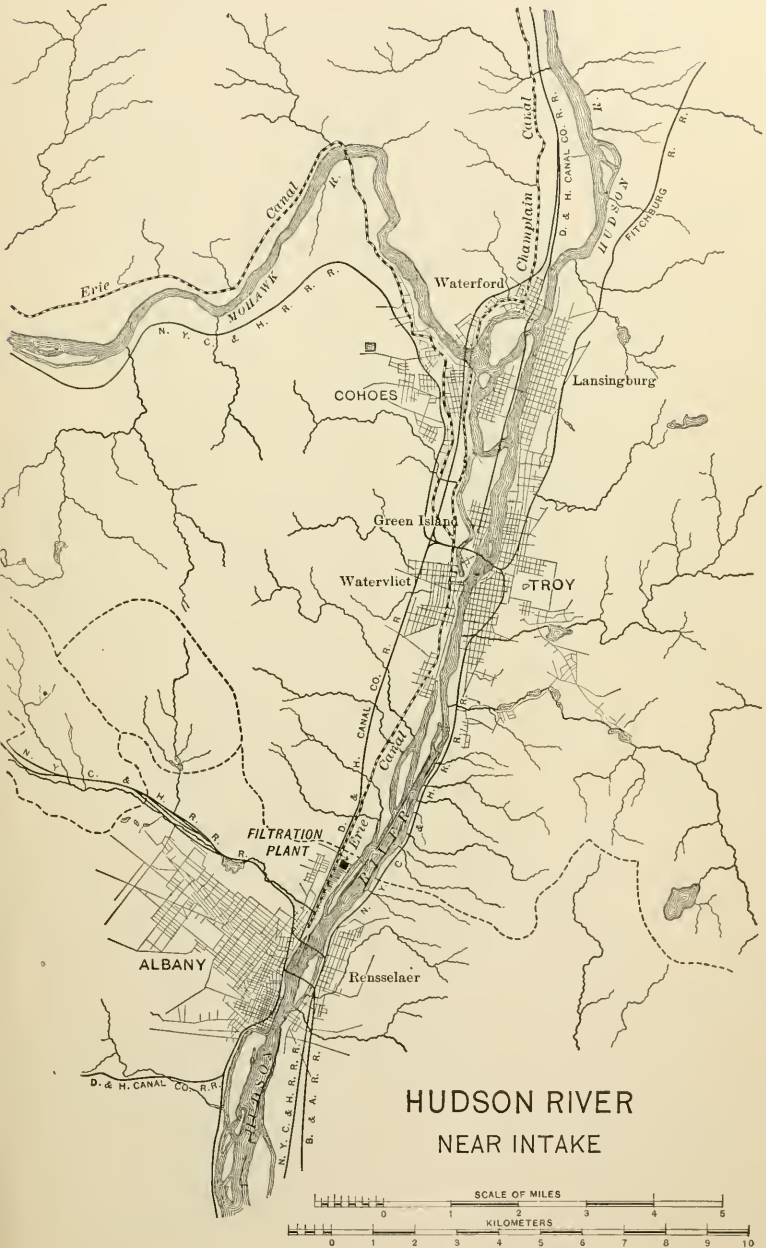

Erie Canal
MOHAWK R.
N. Y. C. & H. R. R. R.
Champlain Canal
D. & H. CANAL CO. R. R.
HUDSON R.
FITCHBURG R. R.
Waterford
COHOES
Lansingburg
Green Island
Watervliet
TROY
D. & H. CANAL CO. R. R.
Erie Canal
N. Y. C. & H. R. R. R.
FILTRATION PLANT
ALBANY
Rensselaer
HUDSON R.
D. & H. CANAL CO. R. R.
N. Y. C. & H. R. R. R.
B. & A. R. R.
HUDSON RIVER
NEAR INTAKE
SCALE OF MILES
0 1 2 3 4 5
KILOMETERS
0 1 2 3 4 5 6 7 8 9 10

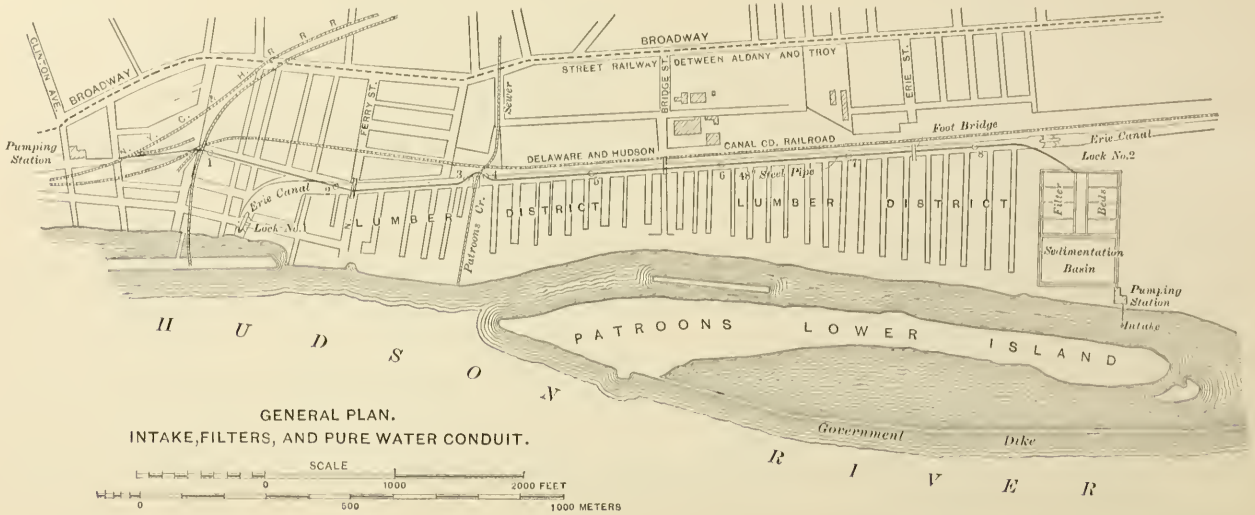
FIG. 2.

TABLE No. 2.—Bacterial Examinations of Water from Main Channel and from Back Channel at Proposed Points of Intake.

Date.	Hour.	Approximate state of tide.	Back Channel.		Main Channel.	
			Turbidity.	Bacteria.	Turbidity.	Bacteria.
1898.						
March 24..	3.00 P. M.	Low tide	.....	2 250	.....	.....
April 13..	11.00 A. M.	Rising	.....	1 927	.....	2 889
20..	7.15 A. M.	High	.....	1 970	.....	.....
20..	8.00 "	High	.....	.....	.....	2 165
June 8..	12.00 M.	High	0.04	400	.....	2 200
20..	1.30 P. M.	Falling	0.09	350	.....	825
24..	7.00 A. M.	Low	0.03	1 600	0.04	1 040
	9.20 "	Rising	0.03	760	0.04	1 120
	11.40 "	Nearly high	0.03	600	0.04	1 040
	2.00 P. M.	After high	0.03	300	0.04	4 000
	4.20 "	Falling	0.03	1 400	0.04	5 200
	6.40 "	Low	0.03	640	0.04	3 600
29..	2.00 P. M.	Nearly high	.....	.....	0.03	2 040
	2.30 "	High	0.04	860	.....	.....
July 7..	9.00 A. M.	High	.....	.....	0.03	4 360
	10.00 "	After high	0.04	560	.....	.....
13..	11.00 A. M.	Rising	.....	.....	0.03	16 780
	12.00 M.	Nearly high	0.03	17 926	.....	.....
21..	11.00 A. M.	Falling	.....	.....	0.03	13 759
	12.00 M.	Falling	0.04	18 724	.....	.....
29..	.....	.....	.....	13 759	.....	15 230
Aug. 4..	.....	.....	.....	20 200	.....	16 070
20..	.....	.....	.....	11 275	.....	7 280
27..	.....	.....	.....	17 286	.....	16 750
Sept. 2..	.....	.....	.....	12 744	.....	9 640
8..	.....	.....	.....	11 860	.....	11 289
13..	.....	.....	.....	14 700	.....	14 366
26..	.....	.....	.....	22 750	.....	17 280
Oct 1..	.....	.....	.....	19 760	.....	16 809
7..	.....	.....	.....	5 260	.....	3 724
21..	.....	.....	.....	9 709	.....	6 240
Nov. 4..	.....	.....	.....	.....	.....	3 560
9..	.....	.....	.....	.....	.....	8 760

Without entering into a detailed discussion, it may be said that the amount of sewage, with reference to the size of the river and the volume of flow, is a fraction less than that at Lawrence, Mass., where a filter plant has also been constructed, but the pollution is much greater than that of most American rivers from which municipal water supplies are taken.

Position of Intake.—The general form of the river channels near the new plant is shown in Figs. 2 and 3. Opposite the plant is a long, narrow island. The land upon which the filter plant is built fronts the back channel of the river. This back channel carried formerly a considerable proportion of the river's flow, but, to improve navigation, the United States Government has constructed a dike, the top of which is about 5 ft. above low water, and this dike cuts off most of the flow



GENERAL PLAN.
INTAKE, FILTERS, AND PURE WATER CONDUIT.

FIG. 3.

through this channel at ordinary river stages. At flood stages the dike is overtopped, and the channel takes a large amount of water.

The question arose as to whether to build an intake across the back channel and the island to the main channel of the river, or to take water from the more convenient back channel. Each point had its advantages. To determine the relative character of the water, examinations were made by Dr. George Blumer, of the Bender Hygienic Laboratory, of Albany. The most important of the results of these examinations are shown in Table No. 2.

The results, up to July 7th, are normal, and represent the condition of the water as it would be ordinarily. Afterward, commencing July 9th, the contractors dumped sand and gravel in the back channel, and took it up again by dredging, for construction purposes, with the result that this water was fouled, and the samples taken after that time do not represent its normal condition.

The results, up to July 7th, showed, in a general way, that the water in the back channel was considerably better than that in the main channel. Occasionally, there was but little difference, and this would always be the case when the river was in flood. At no time was the water in the back channel materially worse than in the main channel.

One of the city sewers enters the river at a point a short distance from the outlet of the back channel, and there was a possibility that sewage therefrom would be carried up the back channel by flood tides. On the other hand, the water in the main channel had come directly from the Troy sewers, while that in the back channel was more or less completely cut off from the main current, and was moved back and forth by the tides, and opportunities for natural purification were present in greater degree than in the main channel. These conditions, apparently, more than offset the possible admixture of fresh sewage.

The plan finally presented was to build two intakes independent of each other, one to each channel, and connected with the pumping station separately; but, in view of the superior average quality of the water in the back channel, up to the time of letting the contract for the intake, it was decided to construct at first only the intake in the back channel, and put in a stub for the other intake, the construction of which was to be deferred until such time as might seem necessary, or indefinitely, should the relative qualities of the raw water in the

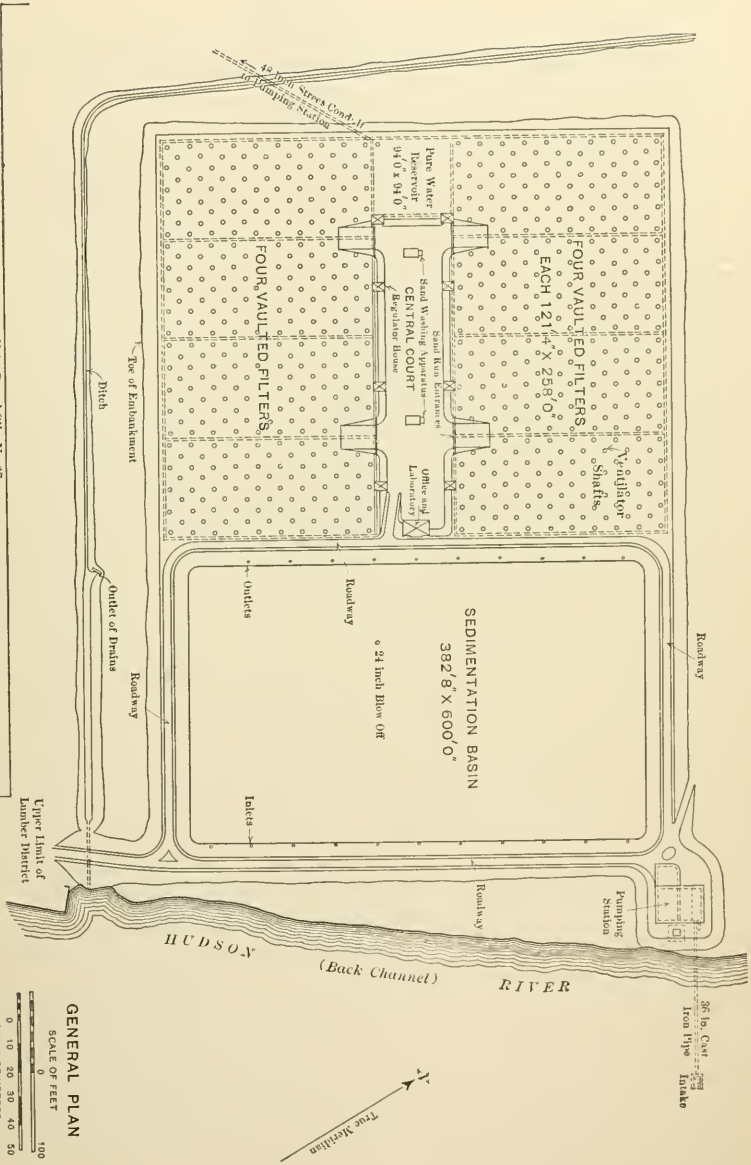

To Main Street Cond. II
Pumping Station
Pure Water Reservoir 94'0" x 94'0"
FOUR VAULTED FILTERS, EACH 121'4" X 258'0"
Ventilator Shafts
Sand Washing Apparatus
Sand Room Entrances
CENTRAL COURT
Regulator House
Office and Laboratory
FOUR VAULTED FILTERS
Toe of Embankment
Ditch
Outlet of Drains
Roadway
SEDIMENTATION BASIN 392'8" X 600'0"
Outlets
Roadway
24 inch Blow Off
Inlets
Roadway
Roadway
Pumping Station
Upper Limit of Lumber District
Erie Canal Slip No. 67
HUDSON (Back Channel) RIVER
36 in. Cast Iron Pipe
Intake
True Meridian
GENERAL PLAN
SCALE OF FEET
0 100
0 10 20 30 40 50
SCALE OF METERS

FIG. 4.

two channels remain as during the time covered by the foregoing examinations.

Description of Plant.

The filter plant, and all structures connected therewith, are shown in Figs. 3 to 15, and this description, for the most part, will be limited to those points which are not thus shown.

Intake.—The intake is shown by Fig. 6, and consists of a simple concrete structure in the form of a box, having an open top covered with rails 6 ins. apart, and connected below, through a 36-in. pipe, with a well in the pumping station. Before going to the pumps the water passes through a screen with bars 2 ins. apart, so arranged as to be raked readily. The rails over the intake and this screen are intended to stop matters which might obstruct the passageways of the pumps, but no attempt is made to stop fish, leaves or other floating matters which may be in the water. The arrangement, in this respect, is like that of the filter at Lawrence, Mass., where the raw water is not subjected to close screening. There is room, however, to place finer screens in the pump well, should they be found desirable.

Pumps.—The centrifugal pumps were built for the Prindle Pump Company at the Lawrence Machine Shop. They have a guaranteed capacity of 16 000 000 galls. per 24 hours against a lift of 18 ft., or 12 000 000 galls. per 24 hours against a lift of 24 ft., corresponding to a water-horse-power, in either case, of 50.5. The ordinary pumping at low water is against the higher lift, and under these conditions either pump can supply the ordinary consumption, the other pump being held in reserve. The plant is arranged, however, so that if for any reason a large quantity of water is required when only one pump can be used, water can be pumped direct to the filters against the lower head, in which case one pump will deliver a larger quantity, up to 16 000 000 galls., the full nominal capacity of the plant.

The boilers were built by James Hunter, of Albany, and are of the vertical tubular type, each of 100 H.-P. The engines are connected directly to the pumps, and were built by the Watertown Engine Company. The plant is supplied with Dean condensers, feed-water filter, and other appurtenances. The whole plant is in duplicate, either half of which is capable of supplying the ordinary consumption, or the consumption up to the limit of capacity of the filters under the above-mentioned conditions.

48" Steel Pipe to Pumping Station
94.0'
Pure Water Reservoir
Sand Washer
FILTER NO. 7
FILTER NO. 8
FILTER NO. 5
FILTER NO. 6
FILTER NO. 3
FILTER NO. 4
FILTER NO. 1
FILTER NO. 2
Division Wall
Inlet
Overflow
Regulator House
Pure Water Pipe
Drain in Regulator House
Sand
Supply to Filters
30" Drain
Laid in Concrete
Outlet to River
Outlet Pipe
Overflow Manhole
Laboratory
Blow Off
SEDIMENTATION BASIN
382.67'
600.0'
Inlet Pipe
Supply from River
4" Supply from City
36" By Pass
Venturi Meter
36" Gate
Pumping Station
Pump Well
Chimney
Intake
PIPE SYSTEMS
DISTRIBUTION OF RAW WATER,
COLLECTION OF PURE WATER,
AND WASTE DRAINS.
SCALE
100 FEET
50 METERS

FIG. 5.

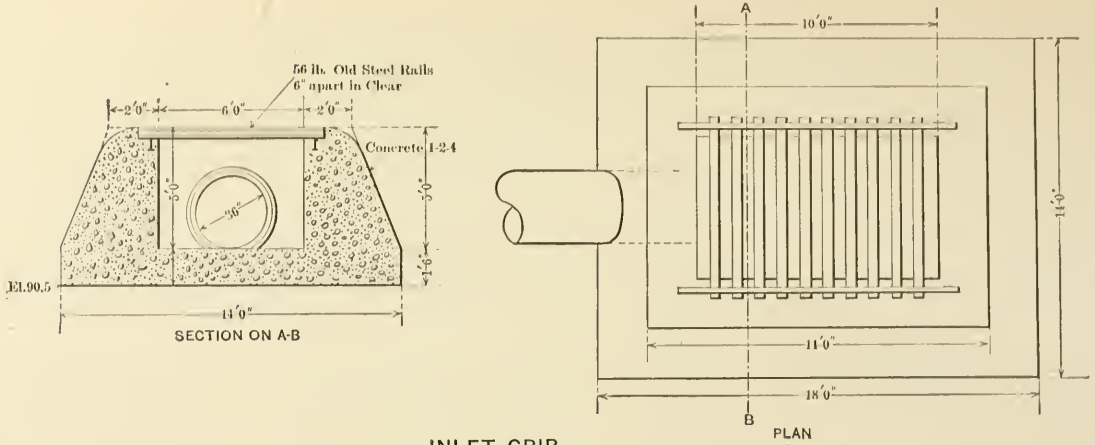
The pumping station building, to a point above the highest flood-level, is of massive concrete construction, without openings. Nearly all the machinery is necessarily below this level, and in high water the sluice gates are closed, and the machinery is thus protected from flooding. The superstructure is of pressed brick, with granite trimmings. The general form of the pumping station and the arrangement of the pumping machinery are shown in Fig. 7. A distant view of the building is shown in Fig. 1, Plate XIV.

Meter for Raw Water.—Upon leaving the pumping station the water passes through a 36-in. Venturi meter having a throat diameter of 17 ins., the throat area being two-ninths of the area of the pipe. The meter records the quantity of water pumped, and is also arranged to show on gauges in the pumping station the rate of pumping.

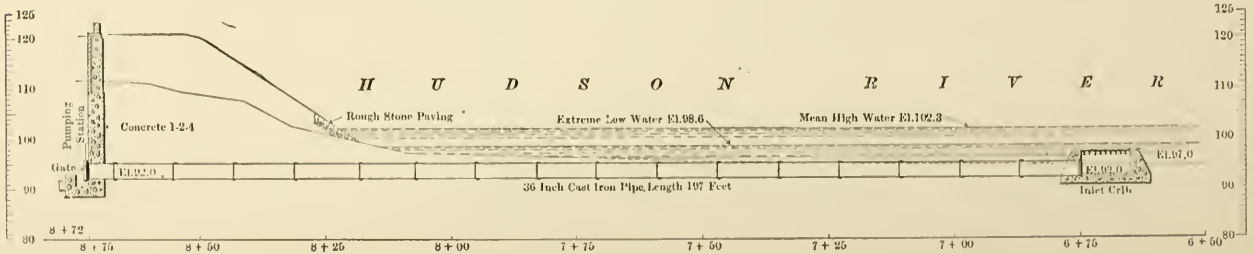
Aeration.—After leaving the meter, the water passes to the sedimentation basin through eleven outlets. These outlets consist of 12-in. pipes on end, the tops of which are 4 ft. above the nominal flow line of the sedimentation basin. Each of these outlet pipes is pierced with 296 $\frac{3}{4}$-in. holes extending from 0.5 to 3.5 ft. below the top of the pipe. These holes are computed so that when 11 000 000 galls. of water per day are pumped, all the water will pass through the holes, the water in the pipes standing flush with the tops. The water is thus thrown out in 3 256 small streams, and becomes aerated. When more than the above amount is pumped, the excess flows over the tops of the outlet pipes in thin sheets, which are broken by the jets.

Regarding the necessity for aeration, no observations have been taken upon the Hudson River, but, judging from experience with the Merrimac, at Lawrence, where the conditions are in many respects similar, the water is at all times more or less aerated, and, for the greater part of the year, it is nearly saturated with oxygen, and aeration is not necessary. During low water in summer, however, there is much less oxygen in the water, and at these times aeration is a distinct advantage. Further, the river water will often have a slight odor, and aeration will tend to remove it. The outlets are arranged so that they can be removed readily in winter, if they are not found necessary at that season.

Sedimentation Basin.—The sedimentation basin has an area of 5 acres and is 9 ft. deep. To the overflow, it has a capacity of 14 600 000 galls., and, to the flow line of the filters, 8 900 000 galls. There is thus



INLET CRIB



PROFILE OF INTAKE PIPE

FIG. 6.

a reserve capacity of 5 700 000 galls. between these limits, and this amount can be drawn upon, without inconvenience, for maintaining the filters in service while the pumps are shut down. This allows a freedom in the operation of the pumps, which would not exist with the water supplied direct to the filters.

The sedimentation basin is built on the river bank, largely above the natural surface of the soil. The sides are embankments made of clay obtained in excavating for the filters, mixed with gravel dredged from the river. These materials were put down in alternate 3-in. layers, wet, and harrowed, and were rolled with 3-ton grooved rollers on the top of each gravel layer until the gravel was forced down into and thoroughly embedded in the clay. The embankments made in this way are extremely solid, stand up in vertical sections when cut, are not readily washed, and no leakage through them has appeared at any point. The outsides of the embankments are covered with soil, and the inside and bottom with 16 ins. of puddle, which is protected from frost on the sides by covering with gravel, above which is a rough blue-stone pavement.

The puddle was made by mixing equal volumes of the clay obtained in excavating for the filters, and mixed sand and gravel obtained from the river by dredging. It differed from the material of the embankments only in more thorough mixing, and greater care in placing. The materials were mixed in a pug mill. It was soon found that the best mixing was secured with rather large quantities of water, while the best ramming required that the materials should not be too wet. Accordingly, the materials were mixed wet, given a preliminary ramming, allowed to stand two or three days, or as long as was necessary, depending upon the weather, and afterward given the final ramming. If the whole became too dry, in the interval, it was moistened at this time. The puddle was put down in three layers, and the concrete rested directly upon it. The concrete was put down in blocks about 7 ft. square, with $\frac{1}{2}$-in. joints, extending half-way from top to bottom, filled with asphalt. The maximum rate of placing puddle was about 3 000 cu. yds. per month.

The water enters the sedimentation basin from eleven inlets along one side and is withdrawn from eleven outlets directly opposite. The inlets and aerating devices described previously bring the water into the basin without current, and evenly distributed along one side.

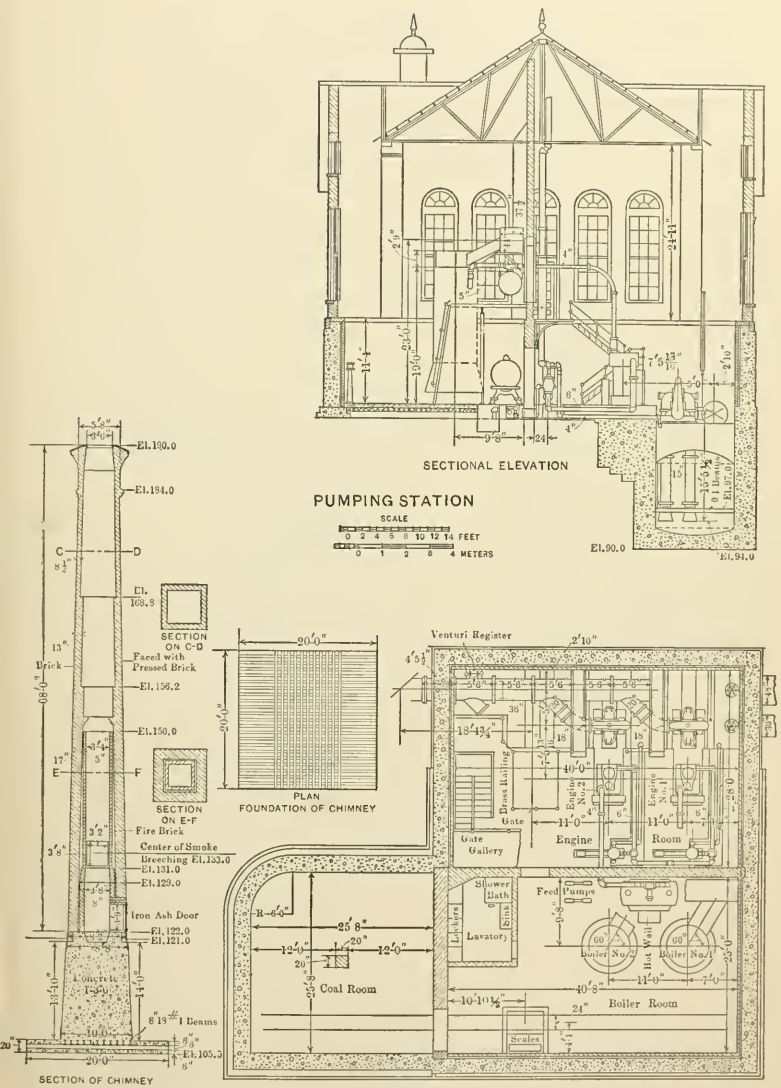

SECTIONAL ELEVATION
PUMPING STATION
SCALE
0 2 4 6 8 10 12 14 FEET
0 1 2 3 4 METERS
El. 90.0
El. 94.0
El. 180.0
El. 184.0
El. 168.8
SECTION ON C-D
Faced with Pressed Brick
El. 156.2
El. 150.0
SECTION ON E-F
Fire Brick
Center of Smoke
Breeching El. 133.0
El. 131.0
El. 129.0
Iron Ash Door
El. 122.0
El. 121.0
Concrete
8" 19# I Beams
El. 105.5
SECTION OF CHIMNEY
PLAN
FOUNDATION OF CHIMNEY
Venturi Register
Engine No. 2
Engine No. 1
Engine Room
Gate Gallery
Gate
Dress Railing
Shower Bath
Sink
Lockers
Lavatory
Feed Pumps
Boiler No. 2
Boiler No. 1
Hot Well
Boiler Room
Coal Room
Scales

FIG. 7.

Both inlets and outlets are controlled by gates, so that any irregularities in distribution can be avoided. The floor of the sedimentation basin is built with even slopes from the toe of each embankment to a sump, the heights of these slopes being 1 ft., whatever their lengths. The sump is connected with a 24-in. pipe leading to a large manhole in which there is a gate through which water can be drawn to empty the basin. There is an overflow from the basin to this manhole, which makes it impossible to fill the basin above the intended level. A section of the embankment about the sedimentation basin and other details are shown in Fig. 8. The method of placing the concrete floors is shown in Fig. 2, Plate XII, while a view of a portion of the finished basin in use is shown in Fig. 1, Plate XIV.

When the basin is being cleaned, the supply is maintained by opening the by-pass from the pumps to the filters and pumping to them direct. Cleaning can be done during periods of continued clear weather, such as occur in the summer and fall, without the slightest detriment to the filters, and it will not be necessary to clean them oftener than once a year.

No special provision is made for flushing out the mud; but, by opening the gates on the inlet and outlet pipes, water can be introduced at twenty-two points along two sides of the basin, and it is believed that this, with the slope of the bottom, will be sufficient to enable the mud to be swept out readily.

Filters.—The filters are of masonry, and are covered to protect them against the winters, which are quite severe in Albany. The piers, cross-walls and linings of the outside walls, entrances, etc., are of vitrified brick. All other masonry is concrete. The average depth of excavation for the filters was 4 ft., and the material at the bottom was usually blue or yellow clay. In some places shale was encountered. In one place soft clay was found, and there the foundations were made deeper.

Floors.—The floors consisted of inverted, groined, concrete arches, arranged to distribute the weight of the walls and vaulting over the whole area of the bottom. The bottoms were put in in alternate squares running diagonally with the pier lines, as shown in Fig. 1, Plate X. In this way forms could be used giving the shape of the arches, and the surface of the concrete was brought to the required form by screeds. After the concrete had set, the forms were removed

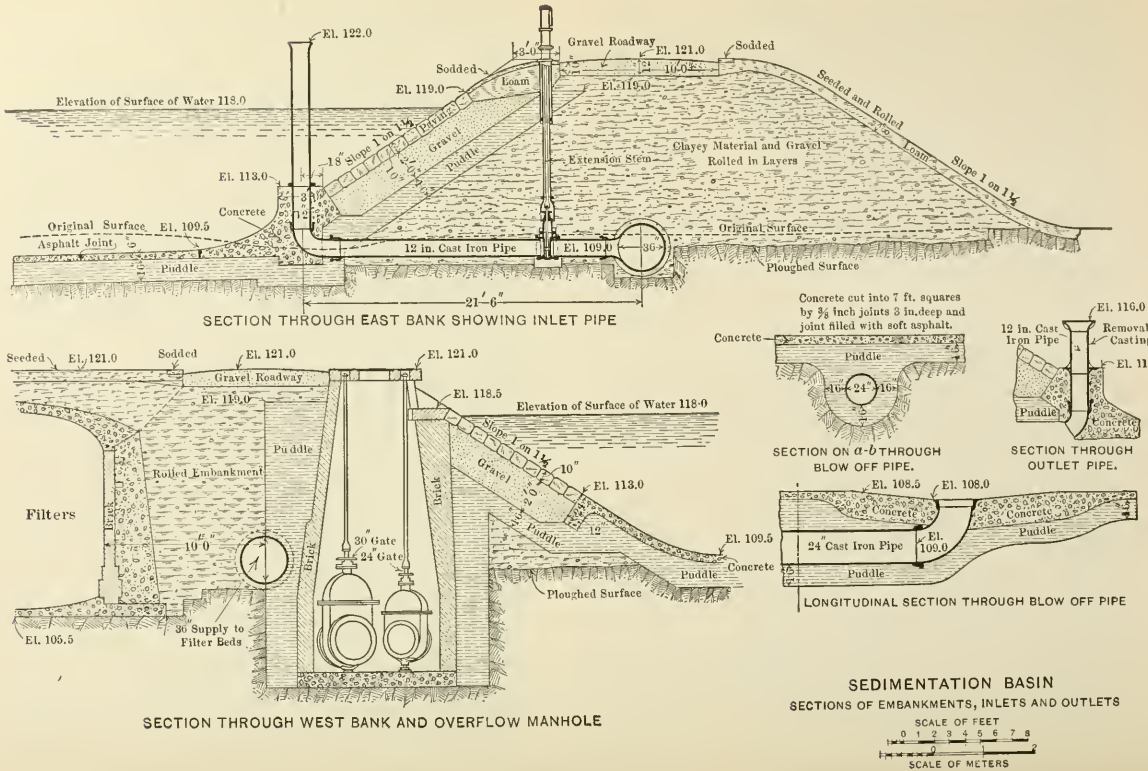

El. 122.0
Gravel Roadway
El. 121.0
Sodded
Loam
El. 119.0
Elevation of Surface of Water 118.0
Gravel
Puddle
Clayey Material and Gravel Rolled in Layers
Extension Stem
Seeded and Rolled Loam
Slope 1 on 1¼
El. 113.0
Concrete
Original Surface
El. 109.5
Asphalt Joint
12 in. Cast Iron Pipe
El. 109.0
Ploughed Surface
21′-6″
SECTION THROUGH EAST BANK SHOWING INLET PIPE
Seeded
Filters
Rolled Embankment
Brick
30″ Gate
24″ Gate
30″ Supply to Filter Beds
El. 105.5
El. 118.5
Slope 1 on 1¼
SECTION THROUGH WEST BANK AND OVERFLOW MANHOLE
Concrete cut into 7 ft. squares by ⅝ inch joints 3 in. deep and joint filled with soft asphalt.
SECTION ON a-b THROUGH BLOW OFF PIPE.
El. 116.0
Removable Casting
SECTION THROUGH OUTLET PIPE.
El. 108.5
24″ Cast Iron Pipe
LONGITUDINAL SECTION THROUGH BLOW OFF PIPE
SEDIMENTATION BASIN
SECTIONS OF EMBANKMENTS, INLETS AND OUTLETS
SCALE OF FEET
SCALE OF METERS

FIG. 8

and the concrete for the intermediate blocks was placed, using screeds to bring the surfaces to the lines given by the blocks already in position. The foundations for the outside walls were put in in alternate sections, the lengths of the sections being usually about 14 ft.

Walls.—For the outside walls the brick linings, 8 ins. thick, were built first to the full height. A certain number of bricks were laid endways, and projected into the concrete. The projecting bricks occupied about 4% of the area of the wall. Afterward, wooden forms were put up on the outside, and the concrete backing was filled in. Sections of the walls are shown in Fig. 10. The arrangement of the projecting brick is shown in Fig. 1, Plate XII, which also shows the outside forms for the concrete wall in the distance.

In stopping the day's work on the outside walls, the concrete was stepped off, the horizontal joints being made at least four times as long as the vertical joints. This bonding proved adequate, and no cracks have followed the joints made in this way. The horizontal joints were strengthened further by driving a wooden stick into the concrete before stopping the work. This was removed afterward, and, when more concrete was added, it formed a thick tongue in the old work. By using sticks, in which deep grooves were cut, and with a slight batter, they could be taken out without trouble and used again indefinitely.

The outside walls are thus practically monolithic, and differ in this respect from the floors and vaulting, which are made in sections, the dimensions of which do not exceed 14 ft.

Vaulting.—The concrete vaulting was placed on wooden centers supported on wedges which could be knocked out after the concrete had set, so that the centers came down readily, and could be moved forward and used again. Some of the centers were used four or five times in the course of the work, the only repairs necessary being the patching of the lagging, and they were in good order at the end of the work. The vaulting was designed with a clear span of 12 ft., a rise of $2\frac{1}{2}$ ft., and a thickness of 6 ins. at the crown, but the clear span was reduced to 11 ft. 11 ins. to fit the sizes of the bricks in the piers. It was put in in squares, the joints being on the crowns of the arches parallel with the lines of the piers, and each pier being the center of one square. The manholes are in alternate sections, and are of concrete, built in steel forms with castings at the tops, securely jointed to the concrete.

FIG. 1.—PLACING THE FLOOR OF A FILTER.

FIG. 2.—BUILDING THE BRICK PIERS.

As the centers for the vaulting were moved forward, it was necessary that they should fit closely the piers and walls in their new positions. The lumber for the centers was cut by machinery, and was of uniform dimensions. Considerable care was required to make sure that the distances between the piers, and between the piers and the walls, should always be exactly right to fit the woodwork. This was accomplished by stopping the work two courses before the top, and giving new lines and grades at this elevation, after which the two upper courses were placed. An opportunity was thus given to correct any slight variations which might have crept in below this level. This procedure was much facilitated on the walls by projecting the upper courses of brick about $\frac{1}{2}$ in., making a slight cornice. Variations of $\frac{1}{4}$ in. or so below this level do not show. Such projections were not used on the piers in this case, but their use would have added to the convenience of construction and the appearance of the finished work.

Above the vaulting there are 2 ft. of earth and soil, grassed on top. The tops of the manholes are 6 ins. above the soil to prevent rain water from entering them. The drainage of the soil is effected by a depression of the vaulting over each pier, partially filled with gravel and sand, from which water is removed by a 2-in. tile drain going down the center of the pier and discharging through its side just above the top of the sand in the filter. The saving in cost by this arrangement was considerable, as the cost of the drains was much less than that of the concrete which would have been necessary to fill the areas over the piers had any other system been adopted. Further, the water entering in this way is as good as any water available, and there is every reason for adding it to the supply. It enters the supply before it passes through the filters. Sections of the vaulting are shown in Fig. 10; details of the centering used are shown in Fig. 13. In Plate XI, Fig. 1 shows the placing of concrete, and Fig. 2 gives a general view of the vaulting at various stages. The finished vaulting is shown from beneath in Plate XIII, Figs. 1 and 2.

In order to provide ready access to each filter, a part of the vaulting near one side is elevated and made cylindrical in shape, making an inclined runway from the sand level to a door, the threshold of which is 6 ins. above the level of the overflow. This sand-run is provided with permanent timber runways and with secure doors.

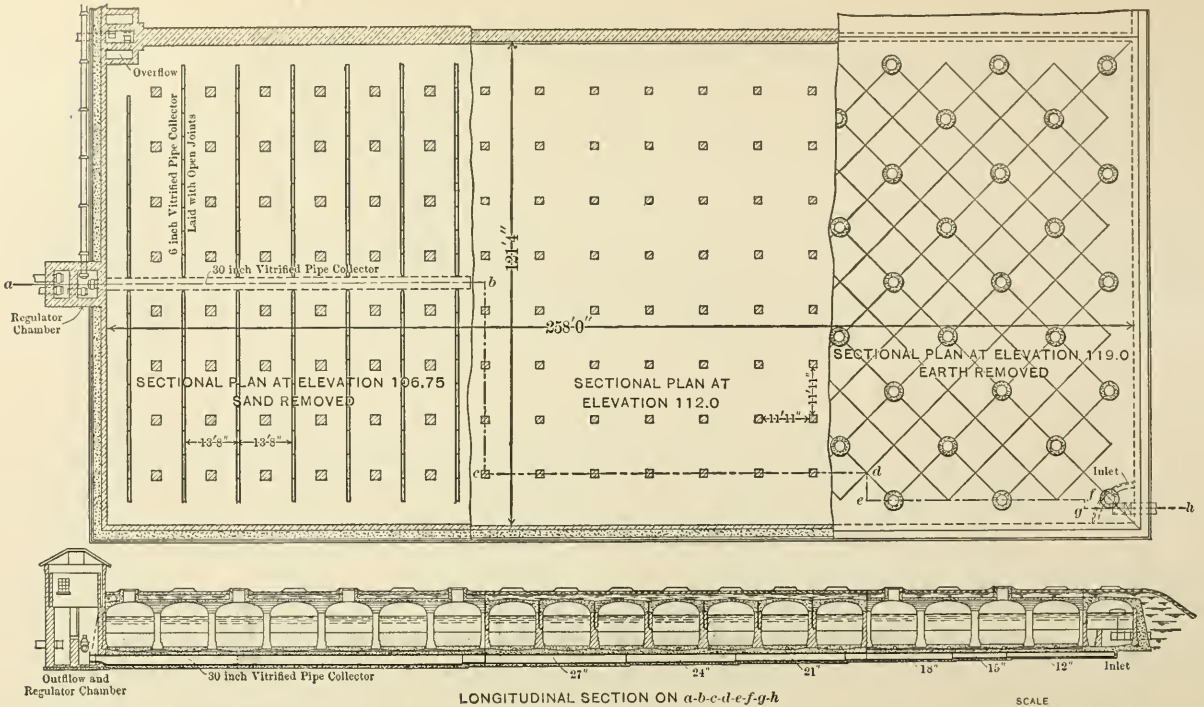
The manholes of the filters are provided with double covers of steel

plates to exclude the cold. The covers also exclude light. When cleaning the filters, light can be admitted by removing the covers. Supports for electric lights are placed in the vaulting, so that the filters can be lighted by electricity and the work of cleaning can be done at night, and in winter under heavy snow, without removing the covers. The electric lights have not yet been installed. The supports for this purpose are castings, which were put at regular places on the centers and surrounded by concrete in the vaulting. They are strong enough, and are placed so as to serve for an overhead carrier system for removing the sand, should that be found advantageous in the future. For the present, the plan is to run out the sand in wheel-barrows, as is commonly done in European filter plants, and thus far in this country. The possibility of putting overhead rails with suspended buckets for carrying out the dirty sand seemed quite promising, but prices secured on the installation of machinery were so great as not to justify it for the removal of the quantity of sand estimated as necessary to be taken out from the filters.

The regulator houses, the entrances to the sand-runs, and all exposed work are of pressed brick, with Milford granite trimmings and slate roofs. The regulator houses have double walls and double windows, and a tight ceiling in the roof, to make them as warm as possible and to avoid the necessity of artificial heat to prevent freezing.

The vaulting is similar, in many respects, to that of the covered filters at Ashland, Wis., and at Somersworth, N. H., but differs from that vaulting, in that it is entirely of concrete instead of brick backed by concrete.

Underdrains.—The main underdrains for removing the filtered water are of vitrified pipe surrounded by concrete, and are entirely below the floors of the filters. These drains were put in before the construction of the filters was commenced, and the concrete surrounding them was brought to the plane of the bottom of the foundations, so that when the floor was built it went over them continuously, without breaking in any way the line of inverted arches. This arrangement was adopted because the drains would have been in the way if they had been placed entirely above the floor, and if any part of the drain had been placed in the normal floor-span it would have reduced the strength of the inverted arches, and would itself have been liable to breakage by their pressure. As the surrounding material is clay



LONGITUDINAL SECTION ON *a-b-c-d-e-f-g-h*

SCALE
0 10 20 30 Feet
0 5 10 Meters

FILTER BEDS

PLAN AND SECTION OF FILTER NO. 2

FIG. 9.

or tight rock, no danger of loss of water by seepage can result from this arrangement.

The main effluent collectors are 30-in. vitrified pipes, reduced by castings to 20 ins. at the outlets, and the effluent from each filter passes through a 20-in. gate. The underdrains are made much larger than would ordinarily be required for carrying the quantities of water involved. The reason for this is that, after a filter has just been scraped, the friction of the water in passing the sand is very slight. If the friction of the underdrainage system is not kept very low, there will be so much loss of head that when a filter is started the pull exerted at remote parts of the filter will be less than at points near the outlet, and thus the parts near the outlet will operate at rates which are too high, while the more remote parts will hardly filter at all, and the resulting purification is less than it should be. The underdrainage system is so designed that, when starting a filter after cleaning, the friction of the sand is about 50 mm. at a rate of 3 000 000 galls. per acre daily, and the friction of the underdrainage system is estimated at 10 mm. This very low friction, which is necessary, is obtained by the use of ample sizes for the underdrains and low velocities in them. In the outlet and measuring devices moderate losses of head are not objectionable, and the sizes of the pipes and connections are, therefore, smaller than the main underdrains.

Connections with the drain are made through thirty-eight 6-in. outlets in each filter, passing through the floor and connected with 6-in. lateral drains running through the whole width of the filter. These drains were made with pipes having one side of the bell cut off, so that they would lie flat on the floor and make concentric joints, without support and without having to be wedged. They were laid with a space of about 1 in. between the barrels, leaving a large opening for the admission of water from the gravel. The general arrangement of the drainage system is shown in Fig. 5. Other details are shown in Figs. 9 and 10, while the computed frictional resistance of one filter is shown in Fig. 11.

Filter Gravel.—The gravel surrounding the underdrains is of three grades. The material was obtained from the river-bed by dredging, and was of the same stock as that used for preparing ballast for the concrete. It was separated and cleaned by a special, cylindrical, revolving screen. The coarsest grade of gravel was that which would not

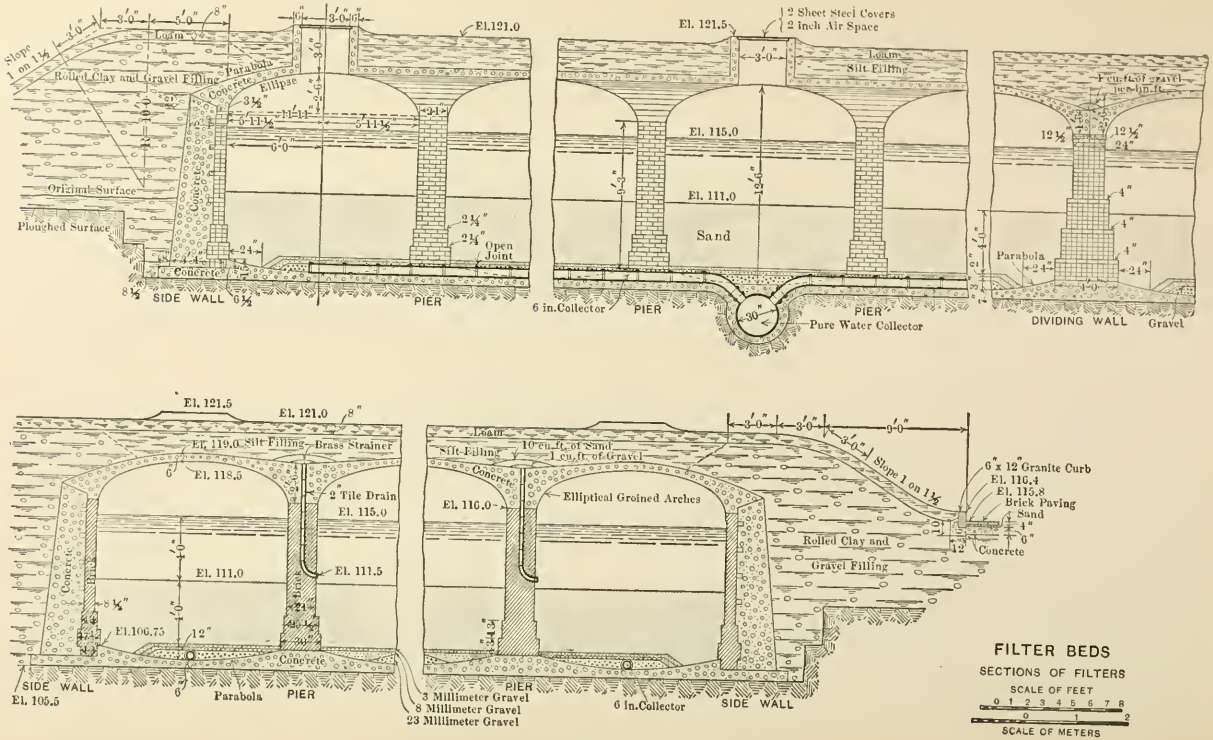

El. 121.0
Loam
Rolled Clay and Gravel Filling
Parabola
Concrete
Ellipse
Original Surface
Ploughed Surface
Open Joint
SIDE WALL
PIER
El. 121.5
2 Sheet Steel Covers
2 inch Air Space
Silt Filling
El. 115.0
El. 111.0
Sand
6 in. Collector
Pure Water Collector
DIVIDING WALL
Gravel
El. 119.0 Silt Filling
Brass Strainer
El. 118.5
2" Tile Drain
El. 115.0
El. 111.5
El. 106.75
SIDE WALL
El. 105.5
Parabola
PIER
10 cu. ft. of Sand
1 cu. ft. of Gravel
Concrete
Elliptical Groined Arches
Slope 1 on 1½
6" x 12" Granite Curb
El. 116.4
El. 115.8
Brick Paving
Sand
Rolled Clay and
Gravel Filling
3 Millimeter Gravel
8 Millimeter Gravel
23 Millimeter Gravel
6 in. Collector
SIDE WALL
FILTER BEDS
SECTIONS OF FILTERS
SCALE OF FEET
SCALE OF METERS

FIG. 10

pass round holes 1 in. in diameter, and free from stones more than about 2 ins. in diameter. At first it was required to pass a screen with holes 2 ins. in diameter, but this screen removed many stones which it was desired to retain, and the screen was afterward changed to have holes 3 ins. in diameter. The intermediate grades of gravel passed the 1-in. holes, and were retained by a screen with round holes $\frac{3}{8}$ in. in diameter. The finest gravel passed the above screens and was retained by a screen with round holes $\frac{3}{16}$ in. in diameter. The gravel was washed, until free from sand and dirt, by water played upon it during the process of screening, and it was afterward taken over screens in the chutes where it was separated from the dirty water, and, when necessary, further quantities of water were played upon it at these points.

The average mechanical analyses of the three grades of gravel are shown by Fig. 14. Their effective sizes were 23, 8 and 3 mm., respectively, and, for convenience, they are designated by those numbers. The average uniformity coefficent for each grade was about 1.8.

The 23-mm. gravel entirely surrounded the 6-in. pipe drains, and was carried slightly above their tops. In some cases it was used to cover nearly the whole of the floor, but this was not insisted upon.

The 8-mm. gravel was obtained in larger quantity than the other sizes, and was used to fill all spaces up to a plane $2\frac{1}{2}$ ins. below the finished surface of the gravel, this layer being about 2 ins. thick over the tops of the drains, and somewhat thicker elsewhere.

The 3-mm. gravel was then applied in a layer $2\frac{1}{2}$ ins. deep, and the surface leveled. The grades for the two upper gravel layers were shown directly by the joints in the brick work of the piers.

The form of construction made it best to put a lateral drain in each section, or 13 ft. 8 ins. apart on centers. The drain itself occupies 7 ins., and the longest course which water has to pass in the gravel, in any event, is about $6\frac{1}{2}$ ft. This distance is so short that the frictional resistance of the filtered water in passing through the gravel is extremely small, and, therefore, it was possible to vary the gravel sections somewhat according to the relative amounts of gravel of the several grades obtained in screening, without detriment to the work. The thickness of the 3-mm. gravel was varied between 2 and $2\frac{1}{2}$ ins., according to the supply available, and similar variations were made in the other grades; but the finished surface of the gravel was always kept

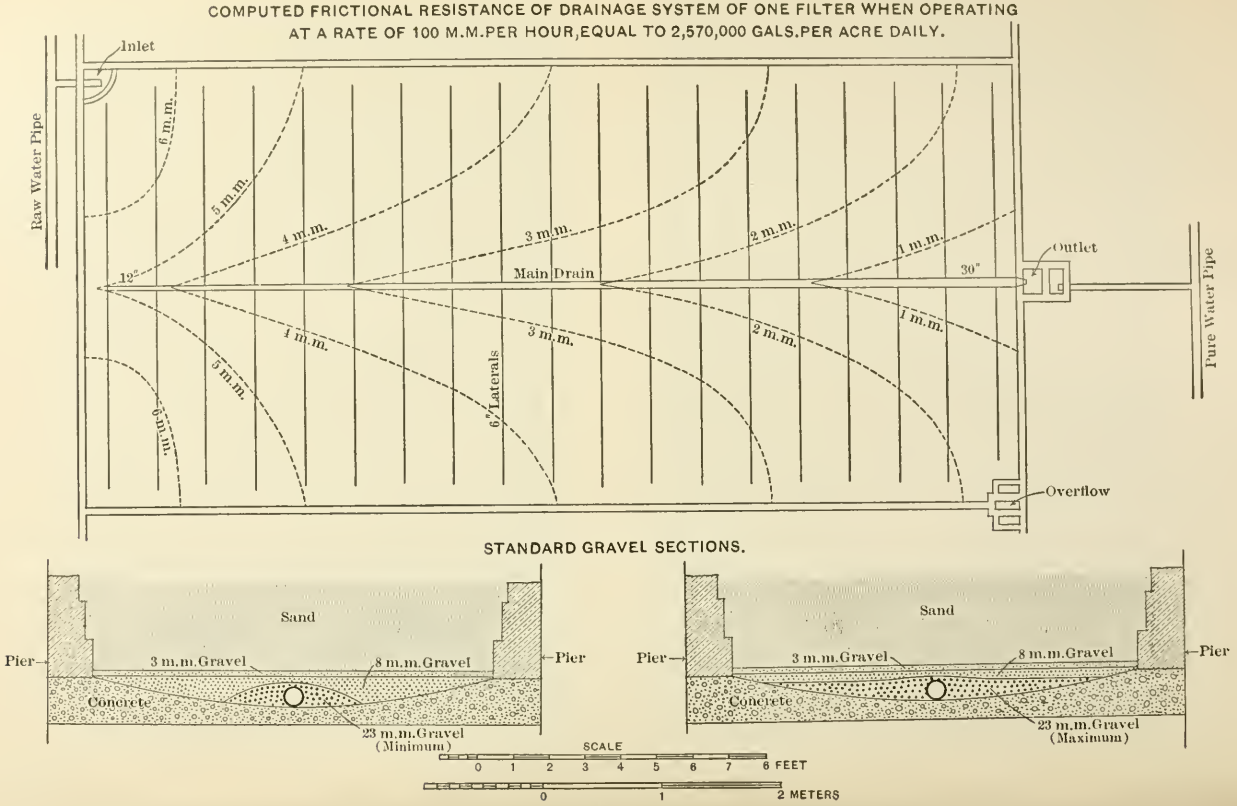

COMPUTED FRICTIONAL RESISTANCE OF DRAINAGE SYSTEM OF ONE FILTER WHEN OPERATING AT A RATE OF 100 M.M.PER HOUR, EQUAL TO 2,570,000 GALS.PER ACRE DAILY.
Inlet
Raw Water Pipe
6 m.m.
5 m.m.
4 m.m.
3 m.m.
2 m.m.
1 m.m.
12"
Main Drain
30"
Outlet
Pure Water Pipe
6" Laterals
Overflow
STANDARD GRAVEL SECTIONS.
Sand
Pier
3 m.m.Gravel
8 m.m.Gravel
Concrete
23 m.m.Gravel (Minimum)
23 m.m.Gravel (Maximum)
SCALE
0 1 2 3 4 5 6 7 6 FEET
0 1 2 METERS

FIG. 11.

at the same elevation. Typical sections of the arrangements of the gravels are shown in Figs. 10 and 11, and in Fig. 1, Plate XIII. The greatest rate at which gravel was obtained and placed was about 1 500 cu. yds. per month.

Filter Sand.—The preliminary estimates of cost were based upon the use of filter sand from a bank near the filter site. Further examination showed that this sand contained a considerable quantity of lime, and it was found by experiment with a small filter constructed for that purpose that the use of this sand would harden the water by about 2 parts in 100 000, and the amount of lime contained in the sand, namely, about 7%, was sufficient to continue this hardening action for a considerable number of years. This was regarded as a serious objection to its use, and the specifications were drawn limiting the amount of lime in the sand. This excluded all of the local bank sands. The river sands which were used were nearly free from lime, and in the end the sand as secured was probably not only free from lime, but more satisfactory in other ways, and also cheaper than the bank sand would have been.

The sand was obtained from the river at various places by dredging. It was first taken up by dipper-dredges, and brought in scows to a point in the back channel a little north of the filter plant. It was there dumped in a specially prepared place in the bottom of the river, from which it was lifted by a hydraulic dredge and pumped through a 15-in. pipe an average distance of 525 ft. to points selected, and varied from time to time, on the flats north of the filters. The water containing the sand was then put through screens having meshes which excluded all stones 5 mm. in diameter and over, and was then taken into basins where the sand was deposited in conical-shaped piles, the water running over the surface and away. The boards around the edges were always kept so low as to prevent ponding and the consequent deposition of fine particles of dirt with the sand. After a certain amount of sand was deposited in one place, the point of discharge was changed, the pile drained, inspected, and, if found satisfactory, carried to the filters.

As the work progressed, an improvement upon this method was made. Piles were driven throughout the area in which the sand was deposited, a bridge was constructed, and the sand trucks passed over it. Sand was shoveled continuously into the trucks, new sand

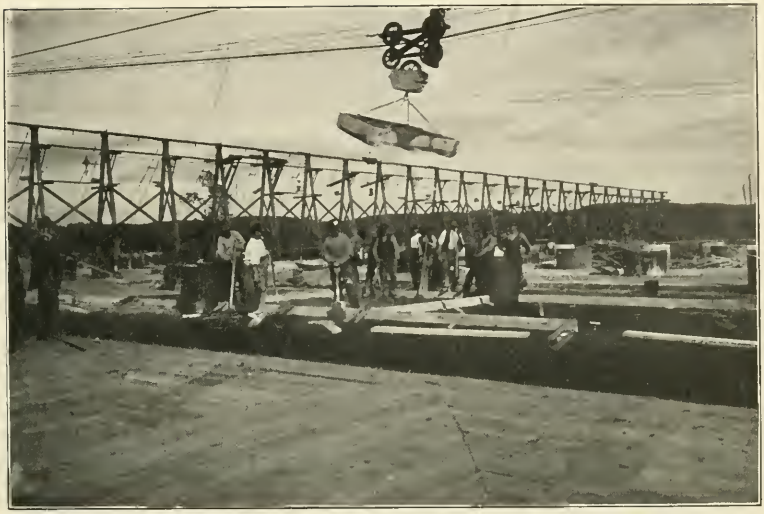


FIG. 1.—PLACING THE CONCRETE VAULTING.

FIG. 2.—GENERAL VIEW OF VAULTING, UNDER CONSTRUCTION.

taking the place of that shoveled out. Afterward, a steam dredge was installed, which took the place of a part of the shovelers. With this arrangement the sand was delivered constantly in one place, and the necessity for changing the position of the screen was avoided.

A considerable amount of the finer part of the sand was removed by this process and accumulated below. This material was not suitable for filter sand and was wasted. Some of it was afterward used for filling. The gravel removed from the sand was used under the pavement of the sedimentation basin, and a part of it was screened for filter gravel. The capacity of the dredge was about 500 cu. yds. per day, scow measurement; but owing to the losses and wastes, and the greater compactness of the sand in the filters, the amount of sand secured, as shown by the final measurements, did not exceed 7 500 cu. yds. per month.

The specifications of the filter sand require that:

"The filter sand shall be clean river, beach or bank sand, with either sharp or rounded grains. It shall be entirely free from clay, dust or organic impurities, and shall, if necessary, be washed to remove such materials from it. The grains shall, all of them, be of hard material, which will not disintegrate, and shall be of the following diameters: Not more than 1% by weight, less than 0.13 mm, nor more than 10% less than 0.27 mm.; at least 10% by weight shall be less than 0.36 mm. and at least 70%, by weight, shall be less than 1 mm., and no particles shall be more than 5 mm. in diameter. The diameters of the sand grains will be computed as the diameters of spheres of equal volume. The sand shall not contain more than 2%, by weight, of lime and magnesia taken together and calculated as carbonates."

With the river sand and the method of handling adopted by the contractors, it was possible to control the quality of the sand, so that the specifications were complied with. In the lower layers of two of the filters a little sand was allowed which contained a few particles above 5 mm. in diameter. The screens were adjusted afterward so that the largest remaining particles were less than 4 mm. in diameter. The filter sand has effective sizes of from 0.29 to 0.32 mm., averaging 0.31; and uniformity coefficients from 2.2 to 2.5, averaging 2.3. Its mechanical composition is shown by the diagram, Fig. 14.

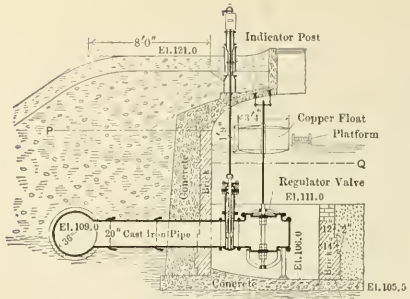
The sand and also the gravel were delivered in the filters through the manholes, temporary plank roadways being built for that purpose. Trucks carrying 1½ yds., with a pair of horses, were driven over the

vaulting in all directions, without hesitation and without damage to it. The sand was dumped on plank platforms constructed below. A record was kept of all the planks used for this purpose, and they were required to be taken up in the presence of inspectors afterward to prevent the possibility of leaving any of them in the filter. This arrangement necessitated working over all the sand underneath the points of dumping, and it thereby became loosened from the excessive packing caused by dumping it from a height. The sand was deposited in three horizontal layers, so that, if by accident sand of unusual quality was placed at any point, the same kind of sand would not extend from top to bottom. The method of placing the sand in layers is shown in Fig. 1, Plate XIII, which also shows the gravel layers and a lateral underdrain; while a completed filter with the sand smoothed ready for use is shown in Fig, 2, Plate XIII.

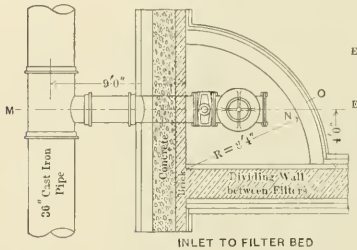
Sand-Washing Apparatus.—Most of the suspended matters in the filtered water are held by the top layer of sand, and this layer is removed from time to time. The dirty sand is washed, and eventually replaced in the filters. Two ejector sand-washing machines, shown in Fig. 15, are provided at convenient places between the filters. In them the dirty sand is mixed with water, and is thrown up by an ejector, after which it runs through a chute into a receptacle, from which it is again lifted by another ejector. It passes in all through five ejectors, part of the dirty water being wasted each time. The sand is finally collected from the last ejector, where it is allowed to deposit from the water.

Sand washers of this kind have been used for many years by some of the London water companies, and more recently at Hamburg; and also at Lawrence and Poughkeepsie in this country. The entire central court between the filters and about the sand washers, shown in Fig. 2, Plate XIV, is paved with brick upon a concrete foundation, and affords a convenient space for handling and storing sand.

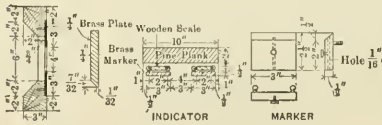
Inlets to Filters.—Water is admitted to each filter through a 20-in. pipe from a pipe system connecting with the sedimentation basin. Just inside of the filter wall is placed a standard gate and beyond that a balanced valve connected with an adjustable float to shut off the water when it reaches the desired height on the filter (Fig. 12). These valves and floats were constructed from special designs, and are similar in principle to valves used for the same purpose in the Berlin water filters.



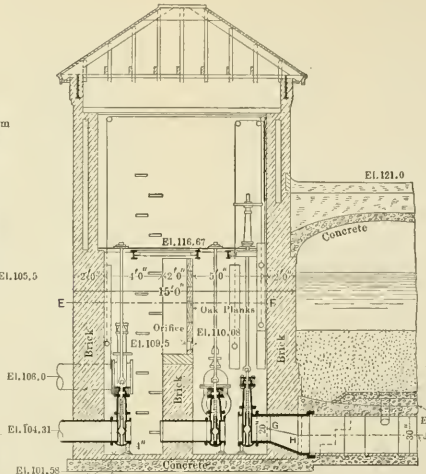
INLET TO FILTER BED. SECTION ON M-N-O



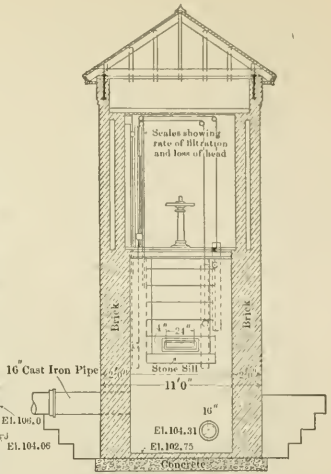
INLET TO FILTER BED
SECTIONAL PLAN ON P-Q



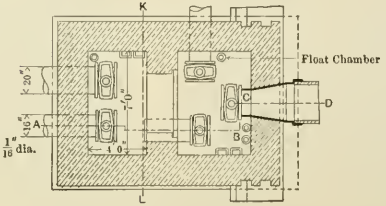
DETAILS OF APPARATUS IN REGULATOR HOUSE



REGULATOR CHAMBER
LONGITUDINAL SECTION ON A-B-C-D



REGULATOR CHAMBER
SECTION ON K-L



REGULATOR CHAMBER
SECTIONAL PLAN ON E-F-G-H-J

FILTER BEDS
INLET VALVES AND REGULATOR CHAMBER

SCALE
0 2 4 6 8 10 FEET
0 1 2 3 4 METERS

FIG. 12.

Overflows.—Each filter is provided with an overflow, so arranged that it cannot be closed, which prevents the water level from exceeding a fixed limit in case the balanced valve fails to act. An outlet is also provided near the sand-run, so that unfiltered water can be removed quickly from the surface of the filter, should it be necessary, to facilitate cleaning.

Filter Outlets.—The outlet of each filter is through a 20-in. gate controlled by a standard graduated to show the exact distance the gate is open. The water rises in a chamber and flows through an orifice in a brass plate 4 by 24 ins., the center of which is 1 ft. below the level of the sand line. At the nominal rate of filtration, 3 000 000 galls. per acre daily, 1 ft. of head is required to force the water through the orifice. With other rates the head increases or decreases approximately as the square of the rate and forms a measure of it. With water standing in the lower chamber, so that the orifice is submerged, it is assumed that the same rates will be obtained with a given difference in level between the water on the two sides of the orifice, as from an equal head above the center of the orifice when discharging into air. The general arrangement of the gate-houses, including the gates for wasting the effluent to the river, for filling filters with filtered water from below, etc., is shown in Fig. 12, while outside views of gate-houses are shown in Fig. 2, Plate X, and Fig. 2, Plate XIV.

Measurement of Effluent.—In order to show the rate of filtration two floats are connected with the water on the two sides of the orifice. These floats are counterbalanced; one carries a graduated scale and the other a marker which moves in front of the scale and shows the rate of filtration corresponding to the difference in level of the water on the two sides. When the water in the lower chamber falls below the center of the orifice, the water in the float chamber is, nevertheless, maintained at this level. This is accomplished by making the lower part of the tube water-tight, with openings just at the desired level, so that when the water falls below this point in the outer chamber it does not fall in the float chamber.

To prevent the loss of water in the float chamber by evaporation, or from other causes, a lead pipe is brought from the other chamber and supplies a driblet of water to it constantly; this overflows through the openings, and maintains the water level at precisely the desired point. The floats thus indicate the difference in water level on the two sides

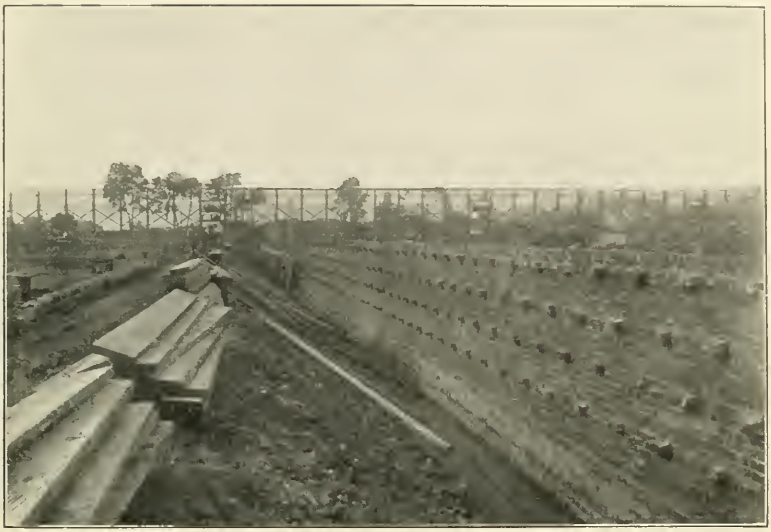


FIG. 1.—OUTSIDE WALL, READY FOR CONCRETE BACKING.

FIG. 2.—SEDIMENTATION BASIN: SHOWING CONSTRUCTION OF FLOOR.

of the orifice whenever the water in the lower chamber is above the center of the orifice; otherwise, they indicate the height of water in the upper chamber above the center of the orifice, regardless of the water level in the lower chamber. The scale is graduated to show the rates of filtration in millions of gallons per acre of filtering area. In computing this scale the area of the filters is taken as 0.7 acre and the coefficient of discharge as 0.61.

At the ordinary rates of filtration the errors introduced by the different conditions under which the orifice operates will rarely amount to as much as 100 000 galls. per acre daily, or one-thirtieth of the ordinary rate of filtration. Usually they are much less than this. The apparatus thus shows directly, and with substantial accuracy, the rate of filtration under all conditions.

Measurement of Loss of Head.—Two other floats with similar connections show the difference in level between the water standing on the filter and the water in the main drain pipe back of the gate, or in other words, the frictional resistance of the filter, including the drains. This is commonly called the loss of head, and increases from 0.2 ft. or less, with a perfectly clean filter, to 4 ft. with the filter ready for cleaning. When the loss of head exceeds 4 ft. the rate of filtration cannot be maintained at 3 000 000 galls. per acre daily with the outlet devices provided, and, in order to maintain the rate, the filter must be cleaned.

Adjustment of Gauges.—The adjustment of the gauges showing the rate of filtration and loss of head is extremely simple. When a filter is put in service the gates from the lower chamber to the pure-water reservoir and to the drain are closed, the outlet of the filter opened, and both chambers allowed to fill to the level of the water on the filter. The length of the wire carrying the gauge is then adjusted, so that the gauge will make the desired run without hitting at either end, and then the marker is adjusted. As both the rate of filtration and loss of head are zero under these conditions, it is only necessary to set the markers to read zero on the gauges to adjust them. The gates can then be opened for regular operation, and the readings on the gauges will be correct. It is necessary to use wires which are light, flexible, and which will not stretch. At first piano wire, No. 27 B. & S. gauge, was used, and was well adapted to the purpose, except that it rusted rapidly. Because of the rusting it was found

necessary to substitute another wire, and cold-drawn copper wire, No. 24 B. & S. gauge, was used with fair results. Stretching is less serious than it would otherwise be, as the correctness of the adjustment can be observed and corrected readily every time a filter is out of service.

From the lower chambers in the regulator houses the water flows through gates to the pipe system leading to the pure-water reservoir. Drain pipes are also provided which allow the water to be entirely drawn out of each filter, should that be necessary for any reason, and without interfering with the other filters, or with the pure-water reservoir.

The outlets of the filters are connected in pairs, so that filtered water can be used for filling the underdrains and sand of the filters from below prior to starting, thus avoiding the disturbance which results from bringing dirty water upon the sand of a filter not filled with water.

Laboratory Building. –The scientific control of filters is regarded as one of the essentials to the best results, and to provide for this there is a laboratory building at one end of the central court between the filters and close to the sedimentation basin, supplied with the necessary equipment for full bacterial examinations, and also with facilities for observing the colors and turbidities of raw and filtered waters, and for making such chemical examinations as may be necessary. This building also provides a comfortable office, darkroom and storage room for tools, etc., used in the work.

Pure-Water Reservoir.—A small pure-water reservoir, 94 ft. square, and holding about 600 000 galls., is provided at the filter plant. The construction is similar to that of the filters, but the shapes of the piers and vaulting were changed slightly, as there was no necessity for the ledges about the bottoms of the piers and walls; while provision is made for taking the rain water, falling upon the vaulting above, to the nearest filters instead of allowing it to enter the reservoir. The floor and roof of the reservoir are at the same levels as those of the filters.

Pure-Water Conduit.—The filtered water is taken from the pure-water reservoir to the present pumping station through 7 913 ft. of 48-in. pipe. For 5 450 ft. the pipe is laid under the Erie Canal, and for 1 837 ft. through Montgomery Street, with an average cut of 22 ft. The pipe is not under pressure, and is made of mild open-hearth steel plates, $\frac{5}{16}$ in. thick, rolled by the Carnegie Steel Company, at Home-

FIG. 1.—INTERIOR OF A FILTER: DRAIN, GRAVEL AND SAND LAYERS.

FIG. 2.—INTERIOR OF A FILTER, READY FOR USE.

stead, Pa. The pipe was made by the Carroll-Porter Boiler and Tank Company, of Pittsburg, and both plates and pipes were inspected by Stowell & Cunningham, of Albany.

Concrete Backing.—Pipe of this thickness is not stiff enough to withstand the earth pressures in the deep cut in Montgomery Street, without being badly deformed, and is not heavy enough to be safe from the danger of floating in the canal, should the water be removed from the pipe for any reason. Concrete was used to support the sides of the pipe in the deep cut and to weight it under the canal. The amount of concrete required for these purposes was considerable, and the Board decided to use a further amount in order to surround the pipe entirely. This makes a concrete pipe with a minimum thickness of 6 ins. outside of the steel pipe for practically the whole distance, capable of supporting the earth, and able itself to serve as a conduit even if the steel pipe should be removed entirely, although in that case it would not be thoroughly water-tight. For 567 ft. on hard clay bottom, the concrete on the bottom was omitted, the natural material being probably equivalent to the concrete. For about 300 ft. at the upper end, in shallow cut and where the pipe is readily accessible, the concrete was also omitted.

The hydraulic gradient of the pipe is 1 in 1 000, and the actual slope is 1 in 1 000 from the pumping station to a point where the depth is limited by the bed of the canal, the top of the pipe being always at least 4 ft. below the nominal bottom of the canal, as required by the canal authorities. It is then level until near the upper end, outside of the canal, where it rises rapidly to the pure-water reservoir.

In passing under the Patroons Creek sewer the canal authorities required that the line should leave the canal and pass under the sewer at one side, returning to the canal afterward, the object being to prevent draining the canal in case of accident to the sewer as a result of the work under it. The section out of the canal was 230 ft. long, and that part of it under the sewer was done in tunnel and below tide level, and was accomplished without accident or injury to the sewer. The entire trench under and near the sewer was back-filled with concrete.

Tightness of Work.—The specifications provided for testing each joint of the pipe under pressure as it was laid. Difficulties were found in carrying this out, and in order to prevent delay to the work, which

would have resulted in serious embarrassment owing to the necessity of filling the canal on May 1st, this test was suspended on part of the line. In place thereof, the pipe was maintained empty after the canal was filled, and all joints were caulked on the inside until perfectly water-tight, and there was no measurable leakage from the canal into the pipe.

Coating.—The pipe was coated by dipping in asphalt, and, after caulking tight on the inside, all imperfections in the coating, due to caulking or otherwise, were covered with melted asphalt in connection with a naphtha lamp. Where the spots were not too large the asphalt on the sides came together when softened by the flame and covered the spot completely without the addition of new material. Where the uncovered patches were large this was not possible. The place was first heated, then melted asphalt was applied and thoroughly heated and melted until it incorporated itself with the old asphalt on the edges.

The greatest difficulty was experienced in repairing breaks in the coating directly on the bottom of the pipe; although these were not serious, owing to the fact that all men who worked therein were required to wear rubbers, and thus the damage done to the coating was slight. Repairs were made by building dams of cotton waste on each side of the defective place and sponging out the water, after which the plates could be heated by a naphtha lamp and repaired in the usual way.

Air Vents.—There are two considerable depressions on the line of the pipe, one under the Bridge Street drain and one under the Patroons Creek sewer. To prevent the air from obstructing the flow of water, the summits on either side of the Bridge Street drain were connected by a 3-in. pipe with expansion joints, going over the top of the sewer and allowing the air to pass by the depression without obstruction even when the pipe is nearly filled with water.

At the Patroons Creek sewer 3-in. pipes are connected with the summits on each side, going up in the manholes to a point above the hydraulic gradient of the water, and are simply left open to the air. To prevent the entrance of dirt, etc., a half-turn neck is screwed on the top, and in the outlet, facing downward, a bushing is put in, reducing the size of the hole, and making it impossible to put in any object which would obstruct the pipe.

FIG. 1.—SEDIMENTATION BASIN, PUMPING STATION AND OUTLETS.

FIG. 2.—CENTRAL COURT, SHOWING DIRTY SAND, FILTER ENTRANCES, ETC.

Connections with Quackenbush Street Pumping Station.—At the lower end, connection is made with the pumping station through a rectangular brick chamber, where the flow of water can be controlled by a 48-in. gate. From this chamber the water is admitted by sluice gates to each of the three wells supplying the Allis pumps, and also to the old pump well. The entrance of water from the river was cut off by digging up the old tunnel, and building a manhole on it, in one side of which was inserted a 36-in. flange pipe and a standard 36-in. gate. The arrangement is such as to cut off the entrance of river water effectively. The gate can be opened at any time, however, if required by any emergency. The gate is accessible, and can be examined for tightness at any time by shutting off the water from the filters and lowering the water in the pump wells. Leakage is further prevented by the fact that the water in the pump wells is normally higher than the water in the river, and the tendency of leakage is outward and not inward.

Cement.—In the construction of the filters and sedimentation basin 30 000 bbls. of Atlas Portland cement were used. Iron Clad Portland cement was used in some parts of this work, and also exclusively on the pumping station and on the pure-water conduit, 8 000 or 9 000 bbls. of it being used in all. Several hundred barrels of other brands of cement were used at times when neither Atlas nor Iron Clad could be obtained promptly. No Rosendale cement was used on any part of the filters, nor in the lower part of the pumping station. On the superstructure of the regulator houses, pump house and laboratory, Rosendale cement and lime were used.

Building Sand.—All the building sand was obtained from the river by dredging, and was screened and washed by machinery. It was of the same general character as the filter sand, but had a higher uniformity coefficient and was not as clean.

Ballast for Concrete.—Gravel, obtained from the river by screening and washing, was used for the most part, but broken stone was used on various parts of the work. Sometimes the materials were used separately, but often they were mixed. The mixture was most satisfactory in every way, making a stronger and more economical concrete than either gravel or broken stone by itself.

Concrete.—The concrete for the filters was mixed by machinery in cubical boxes, 5 ft. cube, and the amount mixed was about 1.6 cu. yds.

at a time. The proportions of mixing were not fixed by the contract, but were left to the discretion of the engineer as the work progressed, and the cement was paid for separately. The usual proportions of mixing were as follows: One barrel of Portland cement, weighing 380 lbs., and nominally assumed to occupy 3.8 cu. ft.; three times this volume of sand, weighing on an average about 90 lbs. per cubic foot, and five times this volume of gravel or ballast, weighing about 100 lbs. per cubic foot; and having an average of 40% of voids. On an average for the whole work 1.26 barrels of cement were used for each cubic yard of concrete, as allowed to the contractors. Actually, the volume of concrete exceeded slightly the nominal dimensions in many cases, and the cement reckoned on all the concrete would be slightly less than the above figure. The discrepancies, however, were not large. The greatest rate of placing concrete on the filters was about 3 700 cu. yds. per month, and, substantially, this rate was maintained for three months.

In placing concrete around, and especially under, the 48-in. steel pipe, difficulty was found in placing that which was made with broken stone. Concrete made from gravel proved much more suitable for this purpose, and a bent rammer facilitated getting it under the pipe. Before the bent rammer was introduced, some difficulty was experienced in getting the contract sections. Afterward they were gotten easily and without trouble.

Brick Work.—All the brick work, except that in the superstructures, was of vitrified paving brick. The brick was not specified, but the specifications required brick absorbing not more than 12% of water by weight. The local brick-makers were unwilling to take the trouble to make brick hard enough to meet this requirement, and the contractors decided finally to use a second-quality paving brick. This was larger than ordinary brick, averaging 8.37 x 3.62 x 2.75 ins., and it was necessary to change the size of the piers in the filters from 20 to 21 ins., the spans of the vaulting and floor arches being reduced to correspond. These changes the contractors were allowed to make, and, in consideration of the superior quality of the brick, they were allowed an amount corresponding to the increased volume of the masonry thereby involved, although this masonry was not believed to be necessary for the stability of the structures. The bricks were regular and uniform in shape, and their use resulted in very satisfac-

tory work. The absorption ranged from 1 to 11%, and averaged 4% by weight.

The mortar used was uniformly 1 part of Portland cement to 3 parts of sand, and in a few cases, where cracks appeared and the masonry was broken, the breaks were through the bricks and not through the joints, although the bricks were extremely hard and strong. The cement was paid for separately, and about 0.80 barrel per cubic yard was used, no deduction being made for ordinary loss and waste. The greatest quantity of brick work laid in one month was 1 100 cu. yds.

Weight on Foundations.—The weight of the finished concrete was about 150 lbs. per cubic foot, as determined by tests. The brick work was computed to weigh 135 lbs. per cubic foot. The weight of the soil filling over the vaulting is estimated at 100 lbs. per cubic foot. Each of the piers in the filters, by itself, weighs 4 300 lbs. The total weight carried by the piers is about 11 tons per square foot, with the soil nearly dry. When the soil is wet, or covered with snow and ice, the weight is increased somewhat.

On the assumption that the inverted concrete arches distribute the weight of the piers over the whole area of the bottom, the weight on the foundations, of all the masonry and the earth cover, but not including the filtering material, amounts to 444 lbs. per square foot, and with the filtering material in place and with water to the nominal flow line, this weight is increased to 1 304 lbs. per square foot.

Assuming the weight of the cross walls to be distributed on the whole width of the monolithic concrete blocks under them, the weight of the masonry, with the earth above, but without the filter sand or water, amounts to 755 lbs. per square foot, and with the sand in position and water to the flow line, to 1 415 lbs. per square foot. The excess weight under the cross-walls thus amounts to 311 lbs. per square foot with the filter empty, and to 111 lbs. per square foot with everything in position.

The foundation was generally on stiff clay capable of carrying much larger loads than the actual ones, but for a small part of the way shale was encountered. It is doubtful if there is any arch action in the floor, in which case the bulk of the weight of the piers is carried on a small area under and near the base, but the arch is there, so that if there is any tendency to settlement at any point it will come into action and

carry the weight. At one place the underlying clay was soft. Apparently, it was the channel of a stream, the position of which had been changed by the Erie Canal, and which was now filled with softer material. The area was so small as not to make it worth while to lower the general elevation of the structures, and extra foundations were provided. These were made by putting concrete, mixed in the proportion of 1: 5: 10, in the form of a wall, from hard bottom up to the ordinary foundation under the walls, and blocks of the same material, 5 ft. square, from the hard bottom up under each pier. These special foundations were put in first, the rest of the material graded to the normal level, and then the floors put in as usual. In these cases the weights on solid foundations are rather more than a ton to the square foot.

The strains in the concrete vaulting, considered as arches, are not heavy, but are somewhat difficult of computation. It appeared probable, after the centers were struck, that the vaulting did not act as arches but as a series of cantilevers. It was put in in squares, each square having a pier for a center, and the blocks were apparently strong enough to carry the loads upon them without coming to an arch action at all. Computed as cantilevers, a tensile strain of 150 lbs. per square inch is sufficient to maintain the loads as they exist, and the strength of the concrete is certainly beyond this figure.

With cold weather, some of the joints in the concrete vaulting opened slightly, indicating the absence of arch action, but no cracks in the blocks have been observed at any point.

Cracks in Walls.—The outside walls of the filters are monolithic concrete, with an 8-in. brick lining. The cross-walls are of brick. Neither have expansion joints of any kind. With cold weather, in December, 1898, cracks were discovered in each of the cross-walls on the north half of the filter plant which was then completed. One crack was found in each wall, and in each case approximately in the middle of the wall. During the winter these cracks opened from $\frac{1}{8}$ to $\frac{3}{8}$ in.

There are also a certain number of cracks on the outside walls, all of which are shown in Fig. 13. There is a break in the continuity of the outside walls where they join the regulator houses. The regulator houses were built first, and the concrete was placed against the brick work afterward, the surface of the brick being left irregular. The brick lining was bonded in, but its 8-in. thickness was not sufficient to

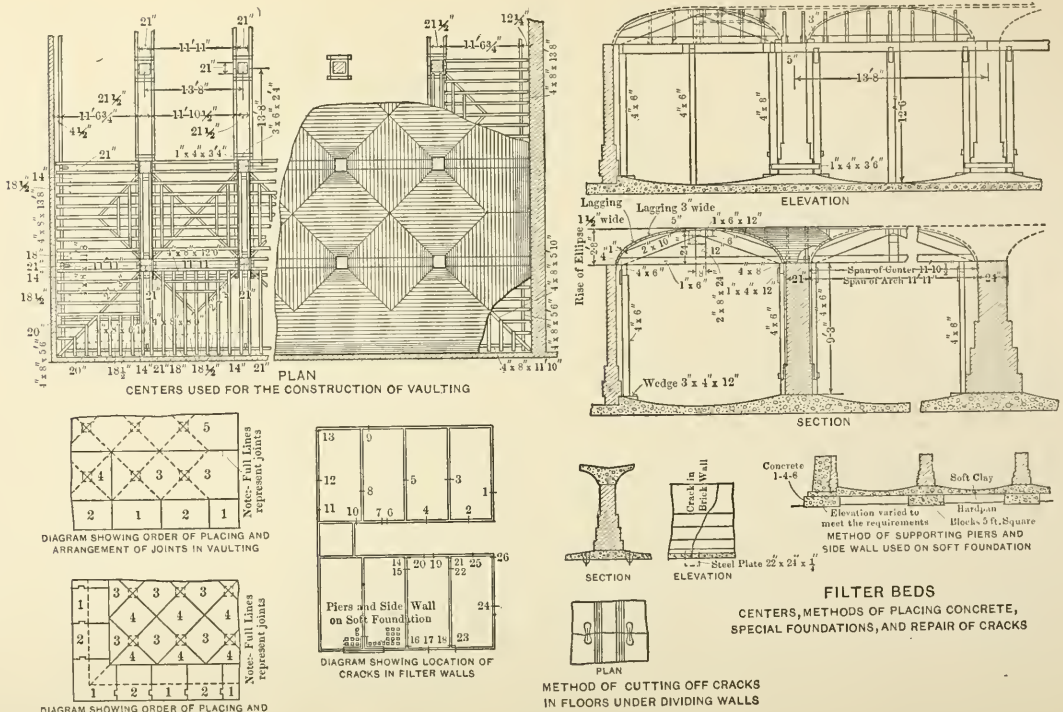

PLAN
CENTERS USED FOR THE CONSTRUCTION OF VAULTING
ELEVATION
SECTION
Lagging 1½" wide
Lagging 3" wide
Rise of Ellipse
Span of Center 11-10½
Span of Arch 11'-11"
Wedge 3" x ¼" x 12"
DIAGRAM SHOWING ORDER OF PLACING AND ARRANGEMENT OF JOINTS IN VAULTING
Notes- Full Lines represent joints
DIAGRAM SHOWING ORDER OF PLACING AND ARRANGEMENT OF JOINTS IN FILTER FLOORS
Notes- Full Lines represent joints
DIAGRAM SHOWING LOCATION OF CRACKS IN FILTER WALLS
Piers and Side Wall on Soft Foundation
SECTION
ELEVATION
Crack in Brick Wall
Steel Plate 22" x 2½" x ¼"
PLAN
METHOD OF CUTTING OFF CRACKS IN FLOORS UNDER DIVIDING WALLS
Concrete 1-4-8
Soft Clay
Hardpan
Elevation varied to meet the requirements
Blocks 5 ft. Square
METHOD OF SUPPORTING PIERS AND SIDE WALL USED ON SOFT FOUNDATION
FILTER BEDS
CENTERS, METHODS OF PLACING CONCRETE, SPECIAL FOUNDATIONS, AND REPAIR OF CRACKS

FIG. 13.

withstand any heavy strain. The walls pulled away from the regulator houses at these points in eight cases, breaking the brick work, although there was no evidence of settlement. The cracks are absolutely without significance, as the clay filling on the outside is quite water-tight. The cracks in the outside walls are also unimportant, for the same reason. Where the outside walls cracked, the crack went approximately in a straight line from top to bottom through both brick and concrete, without following any joints in the concrete wall.

The cracks in the cross-walls were of more importance, for it is possible that unfiltered water might enter them above the sand and follow down through them and enter the gravel and underdrains below without filtration. This action might all take place on one side, or the water might enter the wall above on one side and leave it below on the other. This was rendered more likely, as the floors cracked at the same places and drew apart slightly, and it was actually found that, when the water was on one side of the wall only, it passed through the crack and up through the floor on the other side, some distance away from the wall. To cut this off, a hole was cut in the concrete near the wall, rather more than 2 ft. long and 8 ins. wide, with the ends widened as shown in Fig. 13. The concrete at this point was about 1 ft. thick. A steel plate $\frac{1}{4}$ in. thick and 2 ft. square was placed in the middle of this hole, and driven down until its top was flush with the top of the concrete. It thus projected about 1 ft. into the clay below, and the material was then so hard that the plate had to be severely pounded to get it into position. The hole was then filled with concrete. The steel plate is to serve as an expansion joint. If the crack opens further in the future, the concrete block will break again, approximately at the same place as the original crack, and will slip on the steel plate, the plate remaining across the crack to shut off any possible communication. Any water coming through the wall must thus come to the surface of the floor before reaching the plate, and, to prevent it from getting into the underdrains, the gravel is kept back for 5 ft. at this point, and the sand comes directly to the floor. The water on leaving the crack must then pass through at least 5 ft. of sand before it reaches the underdrains. This will insure, first, a tolerably good purification of such water; and second, if the water is dirty, a gradual clogging up and diminishing of the carrying power of the sand at this point and the final stopping of the leak. In any

event, it will be impossible for an impure water to reach the underdrains. The upper parts of the cracks, which were comparatively unimportant, and were accessible at all times, were caulked with oakum to prevent the passage of water above the sand.

These cracks appear to be clean temperature cracks, with hardly any evidence of settlement. The strength of the wall is such that, with one end free, with the temperature changes, it would pull or shove the wall over the clay without causing other cracks. The only cause of further cracks would be settlement, and this is not regarded as probable. To provide against it, however, as a possible contingency, and also to prevent the possibility of water passing down between the walls and the sand, the gravel is kept 2 ft. away from the walls all the way around, so that in case cracks occur at any points and water passes through them, it will have to pass through 2 ft. of sand. With one crack in each wall, which is virtually an expansion joint, it is not regarded as probable that other cracks will occur which will result in any openings in the floor.

Six cracks were caused by the frost in January and February, 1899. The south part of the filters was left in an uncompleted condition, and, with severe weather, the frost raised the walls in some places as much as 0.2 ft. With warmer weather the walls went back to their proper levels, but the cracks did not close entirely, and some of the brickwork was taken down, while other cracks were caulked with cement.

Without counting the slight openings at the corners of the regulator chambers, where the concrete met the brickwork, nor the six cracks caused by lifting from frost, there were twelve cracks in the whole of the work, and, as the total length of wall was 4 753 ft., there was, on an average, about one crack to each 400 ft. of wall.

Contractors' Plant.—The contractors for the filters and sedimentation basin installed a very elaborate overhead cable-carrier system for the transportation of materials to all parts of the work. This consisted of two trestles 730 ft. apart and over 900 ft. long, between which stretched four cables, each capable of carrying a load of 3 tons, at a height above all the structures on the ground, except the office and regulator houses. These cables were attached to carriages which ran on I-beams on the tops of the trestles, and power was conveyed to both ends by a system of rope drives, so that the cables could be

moved in one direction while the travelers upon the cables were moved in another direction at right angles to it. The trestles for the contractors' plant are shown in Figs. 1 and 2, Plate X, and in Fig. 1, Plate XI, and also in some of the other views. Fig. 1, Plate XI, also shows a skip for carrying concrete, with the carriage upon a cable. The installation of this apparatus required much longer than was anticipated, and delayed the completion of the work very much.

The system was extremely convenient, but its carrying capacity was less than was anticipated, particularly when motion in both directions was required. It was found better after a while to use the carriers, as a rule, in one direction only, and a double-track railroad was built near one of the trestles, which passed the concrete mixer and the gravel screens. Skips were moved on this railroad on trucks, received the concrete and gravel and ran to points opposite the places where the materials were to be used. The skips were then lifted from the trucks by the carrier system and carried to the required places. In this way the lateral motion of the carriers was used only occasionally, as the point at which work was being done changed. The work was so arranged that concrete or gravel was delivered in substantially the same line at one time.

Capacity of Plant and Means of Regulation.

The various filters have effective filtering areas of from 0.702 to 0.704 acre, depending upon slight differences in the thickness of the walls in different places. For the purpose of computation, the area of each filter is taken at 0.7 acre. The nominal rate of filtration is taken as 3 000 000 galls. per acre daily, at which rate each filter will yield 2 100 000 galls. daily, and, with one filter out of use for the purpose of being cleaned, seven filters normally in use will yield 14 700 000 galls. The entrances and outlets are all made of sufficient size, so that rates 50% greater than the foregoing are possible. The capacities of the intake, pumping station and piping are such as to supply any quantity of water which the filters can take, up to an extreme maximum of 25 000 000 galls. in 24 hours. The pure-water conduit from the filters to Quackenbush Street is nominally rated at 25 000 000 galls. per 24 hours, after it has become old and somewhat tuberculated. In its present excellent condition it will carry a larger quantity.

At the pumping station at Quackenbush Street there are three Allis

pumps, each capable of pumping 5 000 000 galls. per 24 hours. In addition to the above there are the old reserve pumps with a nominal capacity of 10 000 000 galls. per 24 hours, which can be used if necessary, but which require so much coal that they are seldom used. For practical purposes the 15 000 000 galls. represents the pumping capacity of this station and also the capacity of the filters, but the arrangements are such that in case of emergency the supply can be increased to 20 000 000 or even 25 000 000 galls. for a short time.

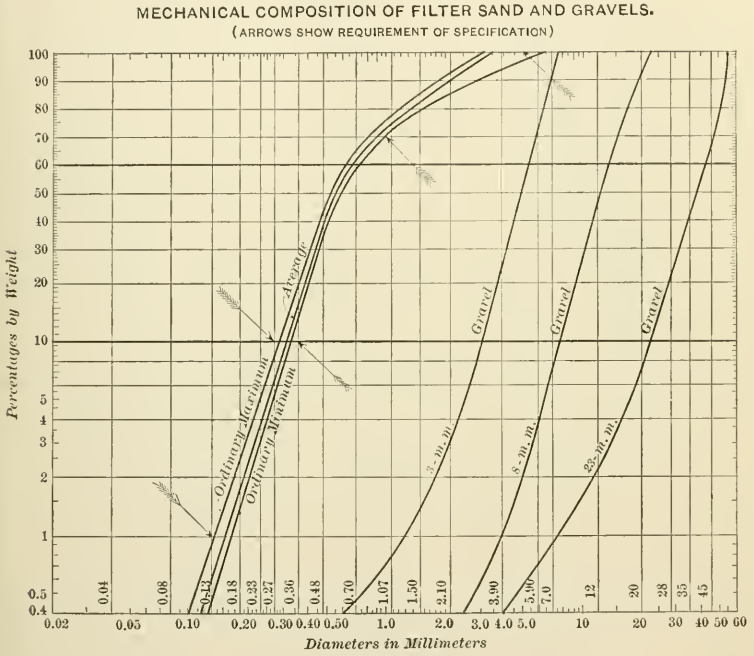


FIG. 14.

The water is pumped through rising mains to reservoirs holding 37 000 000 galls., not including the Tivoli low-service reservoir, which is usually supplied from gravity sources. The reservoir capacity is such that the pumping can be suspended at Quackenbush Street for considerable periods if necessary, and in practice it has been suspended at certain times, especially on Sundays. The amount of water required is also somewhat irregular. The drainage areas supplying the gravity reservoirs are much larger, relatively, than the reservoirs, and at flood periods the volume of the gravity supply is much greater than that

which can be drawn in dry weather. Thus it happens that, at certain seasons of the year, the amount of water to be pumped is but a fraction of the nominal capacity of the pumps, and at these times it is possible to shut the pumps down for greater lengths of time.

Capacity of Pure-Water Reservoir.—The storage capacity provided between the filters and the Quackenbush Street pumps is comparatively small, namely, 600 000 galls., or one hour's supply at the full nominal rate. A larger basin, holding as much as one-third or one-half of a day's supply, would be in many respects desirable in this position, but the conditions were such as to make it practically impossible. The bottom of the reservoir could not be put lower without deepening and increasing greatly the expense of the conduit line. On the other hand, the flow line of the reservoir could not be raised without raising the level of the filters, which was hardly possible upon the site selected. The available depth of the reservoir was thus limited between very narrow bounds, and to secure a large capacity would have necessitated a very large area, and consequently a great expense. Under these circumstances, and especially in view of the abundant storage capacity for filtered water in the distributing reservoirs, it was not deemed necessary to provide a large storage, and only so much was provided as would allow the pumps to be started at the convenience of the engineer, and give a reasonable length of time for the filters to be brought into operation. For this, the pure-water reservoir is ample, but it is not enough to balance any continued fluctuations in the rate of pumping.

Method of Regulating and Changing the Rate of Filtration.—With all the Allis pumps running at their nominal capacity, the quantity of water required will just about equal the nominal capacity of the filters. When only one or two pumps are running, the rate of filtration can be reduced. With the plant operating up to its full capacity, the water level in the pure-water reservoir will be below the level of the standard orifices in the filter outlets. When the rate of pumping is reduced, if no change is made in the gates controlling the filter outlets, the water will gradually rise in the pure-water reservoir and in the various regulator chambers, and will submerge the orifices and gradually reduce the head on the filters, and consequently the rates of filtration, until those rates equal the quantity pumped. In case the pumping is stopped altogether, the filters will keep on delivering at gradually

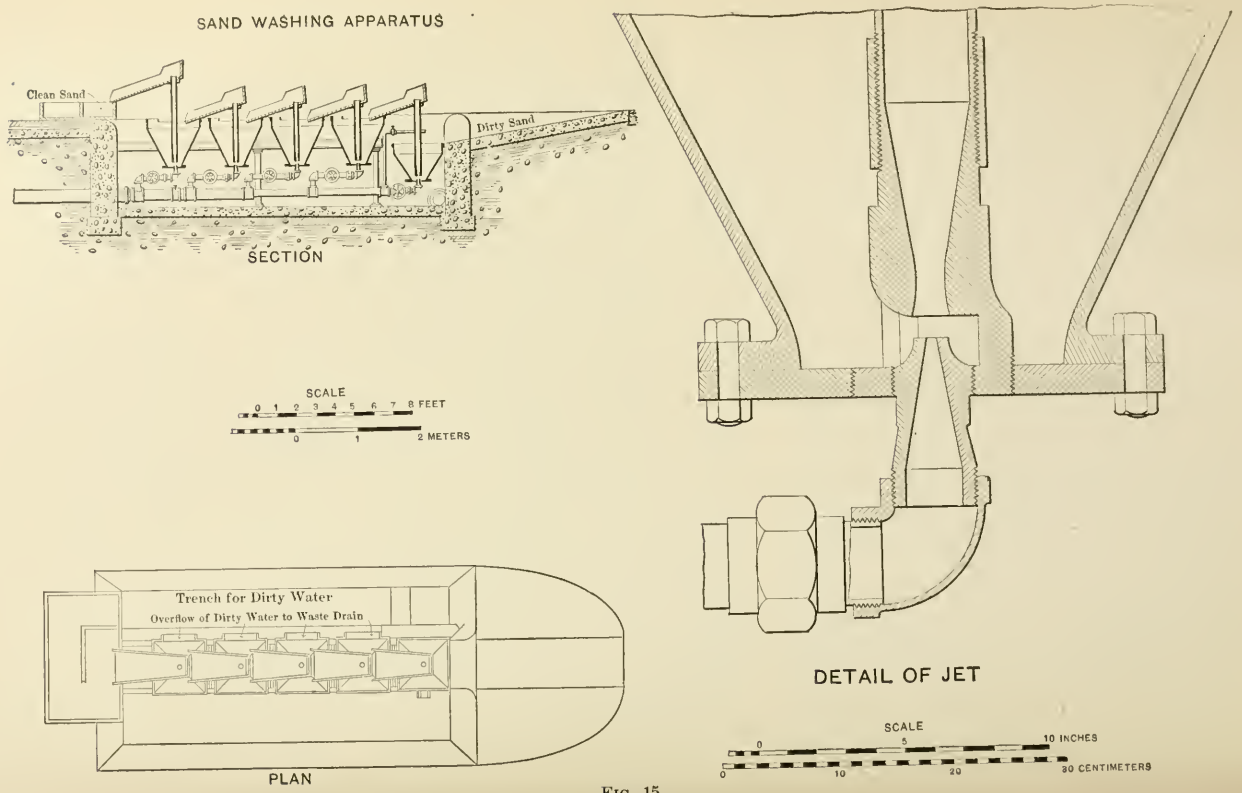

SAND WASHING APPARATUS
Clean Sand
Dirty Sand
SECTION
SCALE
0 1 2 3 4 5 6 7 8 FEET
0 1 2 METERS
Trench for Dirty Water
Overflow of Dirty Water to Waste Drain
PLAN
DETAIL OF JET
SCALE
0 5 10 INCHES
0 10 20 30 CENTIMETERS

FIG. 15.

reduced rates until the water level in the pure-water reservoir reaches that of the water on the filters.

When the pumps are started up, after such stoppage or reduced rate of pumping, the water levels in the pure-water reservoir and in the gate chambers will be lowered gradually, and the filters will start to operate it first with extremely low rates which will increase gradually until the water is depressed below the orifices, when they will again reach the rates at which they were last set. The regulators during all this time will show the rate of filtration on each filter, and, if any inequalities occur which demand correction, the gates on the various outlets can be adjusted accordingly.

The arrangement, in this respect, combines some of the features of the English and German plants. In the English plants the filters are usually connected directly with the clear-water basin, and that in turn with the pumps, and the speed of filtration is required to respond to the speed of the pumps, increasing and decreasing with it, being regulated at all times by the height of water in the pure-water reservoir. This arrangement has been subject to severe criticism, because the rate of filtration fluctuates with the consumption, and especially because the rates of filtration obtained simultaneously in different filters may be different. There was no way to determine at what rate any individual filter was working, and there was always a tendency for a freshly scraped filter to operate much more rapidly than those which had not been scraped for some time.

This led to the procedure, first formulated by the Commission of German Water-Works Engineers in 1894, and provided for in most of the German works built or remodeled since that time, of providing pure-water storage sufficient in amount to make the rate of filtration entirely independent of the operation of the pumps. Each filter was to be controlled by itself, be independent of the others, and deliver its water into a pure-water reservoir lower than itself, so that it could never be affected by back-water, and so large that there would never be a demand for sudden changes in the rate of filtration.

This procedure has given excellent results in the German works; but it leads oftentimes to expensive construction. It involves, in the first place, a much greater loss of head in passing through the works, because the pure-water reservoir must be lower than the filters, and the cost of the pure-water reservoir is increased greatly

because of its large size. The regulation of the filters is put upon the attendants entirely, or upon automatic devices, and regulation by what is known as "responding to the pumps" is eliminated.

More recently, the German authorities have shown less disposition to insist rigidly upon the principles advanced in 1894. In a compilation of the results of several years' experience with German water filters, Dr. Pannwiz,* makes a statement of particular interest, of which a free translation is as follows:

"Most of the German works have sufficient pure-water reservoir capacity to balance the normal fluctuations in consumption, so that the rate of filtration is at least independent of the hourly fluctuations in consumption. Of especial importance is the superficial area of the pure-water reservoir. If it is sufficiently large there is no objection to allowing the water level in it to rise to that of the water upon the filters. With very low rates of consumption during the night the filters may work slowly and even stop, without damage to the sediment layers when the stopping and starting take place slowly and regularly, because of the ample reservoir area.

"The very considerable fluctuations from day to day, especially those arising from unusual and unforeseen occurrences, are not provided for entirely by even very large and well-arranged reservoirs. To provide for these without causing shock, the rate of filtration must be changed carefully and gradually, and the first essential to success is a good regulation apparatus."

"Responding to the pumps" has a great deal to recommend it. It allows the pure-water reservoir to be put at the highest possible level, it reduces to a minimum the loss of head in the plant, and yet provides automatically, and without the slightest trouble on the part of the attendants, for the delivery of the required quantity of water by the filters at all times. If the filters are connected directly to the pumps there is a tendency for the pulsations of the pumps to disturb their operation, which is highly objectionable, even if the pumps are far removed; and this exists where filters are connected directly to the pumps, and a pure-water reservoir is attached to them indirectly. By taking all the water through the pure-water reservoir and having no connection except through it, this condition is absolutely avoided, and the pull on the filters is at all times perfectly steady.

Much has been said as to the effect of variation in the rate of filtration upon the efficiency of filters. Experiments have been made

*"*Arbeiten aus dem Kaiserlichen Gesundheitsampte*," Vol. xiv., p. 260.

at Lawrence and elsewhere which have shown that, as long as the maximum rate does not exceed a proper one, and under reasonable regulations, and with the filter in all respects in good order, no marked decrease in efficiency results from moderate fluctuations in rate. There is probably a greater decrease of efficiency by stopping the filter altogether, especially if it is done suddenly, than by simply reducing the rate. The former sometimes results in loosening air bubbles in the sand, which rise to the surface and cause disturbances, but this is not often caused by simple change in rate.

On the whole, there is little evidence to show that, within reasonable limits, fluctuations in rate are objectionable, or should be excluded entirely, especially in such cases as at Albany, where arrangements to prevent them would have resulted in very greatly increased first cost. The inferior results sometimes obtained with the system of "responding to the pumps" as it existed in earlier works, and still exists in many important places, undoubtedly arises from the fact that there is no means of knowing and controlling the simultaneous rate of filtration in different filters, and that one filter may be filtering two or three times as fast as another, with nothing to indicate it.

This contingency is fully provided for in the Albany plant. The orifices are of such size that even with a filter just scraped and put in service, with the minimum loss of head, with the outlet gate wide open, and with the water level in the pure-water reservoir clear down; that is, with the most unfavorable conditions which could possibly exist, the rate of filtration cannot exceed 5 000 000 or 6 000 000 galls. per acre daily, or double the nominal rate. This rate, while much too high for a filter which has just been cleaned, is not nearly as high as was possible, and in fact actually occurred in the old Stralau filters at Berlin, and in many English works; and, further, such a condition could only occur through the gross negligence of the attendants, because the rate of filtration is indicated clearly at all times by the gauges. These regulating devices have been specially designed to show the rate with unmistakable clearness, so that no attendant, however stupid, can make an error by an incorrect computation from the gauge heights. It is believed that the advantage of clearness by this procedure is much more important than any increased accuracy which might be secured by refinements in the method of computation,

which should take into account variations in the value of the coefficient of discharge, but which would render direct readings impossible.

In designing the Albany plant the object has been to combine the best features of German regulation with the economical and convenient features of the older English system, and filters are allowed to respond to the pumps within certain limits, while guarding against the dangers ordinarily incident thereto.

Results of Operation.

The filters were designed to remove from the water the bacteria which cause disease. They have already reached a bacterial efficiency of over 99%, and it is expected that their use will result in a great reduction in the death rate from water-borne diseases in the city. They also remove a part of the color and all of the suspended matters and turbidity, so that the water is satisfactory in its physical properties.

The filters have reached, with perfect ease, their rated capacity, and on several occasions have been operated to deliver one-third more than this amount; that is to say, at a rate of 4 000 000 galls. per acre, daily.

The filters are operated by George I. Bailey, M. Am. Soc. C. E., Superintendent of the Albany Water-Works, whom the writer hopes will present further particulars concerning the cost and results of operation.

Cost of Construction.

The cost of the filtration plant complete is shown by Table No. 3. As some of the accounts are not yet closed, a few of the items, designated by stars (*) are estimated, and are subject to slight change with the closing of all the accounts.

The filters, sedimentation basin and pure-water reservoir are connected in such a way as to make an exact separation of their costs impossible; but, approximately, the sedimentation basin cost $60 000, the pure-water reservoir $9 000, and the filters $255,000. The sedimentation basin thus cost $4 100 per million gallons capacity; and the filters complete, including all piping, cost $45 600 per acre of net filtering area, exclusive of land and engineering.

TABLE No. 3.—Approximate Cost of Filtration Plant Complete.

Land		$8 290.00
Pumping Station:		
Pumping machinery, boilers, etc.	$22 000.00	
Intake	3 800.00	
Pump well, gates, screens and foundations	6 285.00	
Pumping station building	12 167.00	
Chimney	1 540.00	
Venturi meter	1 560.00	
Extra work and minor items	2 393.00*	
		49 745.00*
Filters, Sedimentation Basin and Pure-Water Reservoir:		
Preliminary draining	$1 956.71	
70 672 cu yds. excavation, at (average) $0.3079 +	21 761.64	
16 040 cu. yds. rolled clay and gravel embankment, at $0.52	8 340.80	
22 400 cu. yds. silt and loam filling, at $0.15	3 360.00*	
22 000 cu. yds. general filling rolled, at $0.18	3 960.00*	
12 550 cu. yds. puddle, at $0.715	8 973.25	
1 775 cu. yds. gravel for lining, at $0.85	1,508.75	
2 257 sq. yds split stone lining, at $0.82	1 850.74	
11 737 cu. yds. concrete in floors, at $2.31	27 112.47	
7 792 cu. yds. concrete in vaulting, at $3.85	29 999.20	
3 147 cu. yds. all other concrete, at $2.13	6 703.11	
4 382 cu. yds. brick work, at $8.125	35 603.75	
31 715 barrels Portland cement, at $1.935	61 368.53	
7 281 cu. yds. filter gravel, at $1.05	7 645.05	
36 100 cu. yds. filter sand, at $1.00	36 100.00*	
Cast-iron pipes and specials, placed, including placing gates	21 841.25	
Gates and valves	6 714.00	
Vitrified pipe, complete	7 153.32	
672 filter manhole covers, at $4.40	2 956.80	
8 sand-run fixtures, at $407.50	3 260.00	
8 regulator houses, at $862.24	6 897.92	
1 office and laboratory	4 881.00	
Vitrified brick paving	2 158.00	
Iron fence about court	1 704.00	
Extra work and all minor items	10 150.11*	
		323 960.40*
Conduit and Connections with Quackenbush Street Pumping Station:		
7 913 ft. 48-in. steel pipe, at $4.50	$35 608.50	
24 218 cu. yds. excavation at (average) $0.855 +	20 715.60	
3 144 cu. yds. concrete, at $5.00	15 720.00	
Gates and connections with pumping station	3 680.00	
Sewer and railroad crossings, sheeting, and all other items	10 914.12	
		86 638.22
Engineering, inspection, printing, etc.		28 000.00*
Total approximate cost of work		$496 633.62*

The preliminary estimate of cost submitted to the Board, February 8th, 1897, amounted to $478 000. The work will actually cost, in round numbers, $20 000, or 4% more than the preliminary estimate.

The preliminary estimate, and the actual costs by more important divisions, are as follows:

	Preliminary estimate, Feb. 8, 1897.	Approximate actual cost.
Land	$10 000	$8 290
Pumping station and intake	34 000	49 745
Filters and sedimentation basin, with piping	327 000	323 960
Pure-water conduit and connection, with Quackenbush Street pumping station	64 000	86 638
Engineering and contingencies	43 000	28 000
Total	$478 000	$496 633

The excess in the cost is in the pumping station and conduit line. The cost of the conduit line was increased by surrounding it with concrete, which was not included in the preliminary estimate, and also by certain changes in the location required by the canal authorities. The pumping station also was made more elaborate and expensive than was contemplated in the preliminary estimate. Otherwise the work was executed substantially as first planned.

Acknowledgment.

The general plan and location of the plant were first conceived by the Superintendent of Water-Works, George I. Bailey, M. Am. Soc. C. E., and the successful execution is largely due to his efforts. The members of the Water Board, and especially the Construction Committee, have followed the work in detail closely and personally, and their interest and support have been essential factors in the results accomplished. In the designs and specifications for the pure-water conduit the author is greatly indebted to Emil Kuichling, M. Am. Soc. C. E., and also for most valuable suggestions relative to the performance of this part of the work. To William Wheeler, M. Am. Soc. C. E., of Boston, the author is indebted for advice upon the vaulting and cross-sections of the walls, and these matters were submitted to him before the plans were put in final shape. All the architectural designs have been supplied by Mr. A. W. Fuller, of Albany. W. B. Fuller, M. Am. Soc. C. E., as Resident Engineer, has been in direct charge of the work, and its success is largely due to his interest in it and the close attention which he and the assistant engineers have given it.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

INSUFFICIENT PROVISION FOR COUNTER-STRESSES IN RAILROAD BRIDGES.

Discussion.*

By Messrs. F. C. KUNZ, IRA O. BAKER and HENRY S. PRICHARD.

Mr. Kunz. F. C. KUNZ, M. Am. Soc. C. E. (by letter).—The fact that the counters are the weakest members of a bridge is justly claimed as one of the reasons for the preference of those specifications which, contrary to the fatigue formulas, assume a constant unit stress and add to the live load a certain percentage for the effect of impact. This percentage varies inversely with the loaded length, and is, therefore, greater for the counters than for the main diagonals, and greater for the latter than for the chords. The result is that the counters are comparatively heavier than the other members of the truss, and are sometimes necessitated in panels in which the fatigue formulas would not call for their use. It is evident that a portion of this percentage for impact can be considered as an increase of the live load itself, as long as we reduce the impact, for instance, by reducing the velocity of the live load, thus bringing the latter nearer to a static loading. The result would be, however, that the different members of the bridge would be subjected to different unit stresses. This is allowable for small variation of the live load, but, in the last few years, the increase in the weight of the rolling load has been extraordinary, as the following figures will show. In the beginning of this decade a great number of bridges were in service which were designed originally for a live load consisting of two engines of 55 tons each, followed by cars weighing 20 tons each, while the new Specifications of that time called

* Continued from October, 1899, *Proceedings*.

for a live load consisting of 100-ton engines and cars of 40 tons. At the present time, most railroads specify engines weighing 120 up to 175 tons (Lehigh Valley Mountain Engine) and cars weighing up to 50 tons, and more, and there is no indication that the limit has been reached. Mr. Kunz.

In 1895, when the Pencoyd Iron Works were building a great number of bridges for the Missouri, Kansas and Texas Railway, they allowed in their designs—"Pencoyd Standards"—for a future increase of the live load in the following manner:

The unit stress of the Pencoyd Specifications for soft steel is 15 000 lbs. per square inch, but there is no reason why the material could not be stressed to two-thirds of the elastic limit, *i. e.*, 20 000 lbs. per square inch if necessary. It was determined what increase of live load, considering the impact, corresponded to an allowable unit stress of 20 000 lbs. per square inch in the chords. This increase depends, of course, on the ratio of the dead load to the sum of the live load and the percentage for impact, and is a minimum when the dead load = 0, in which case it is 33 per cent. The chords were selected because of their considerable weight, amounting to about one-half of the whole truss, and because they are, of all the truss members, affected the least by an increase of the live load, since the ratio of the approximately constant dead-load stress to the stress caused by the varying live load is the greatest. For ordinary plate-girders the allowable increase was found to be 35 to 38%; for a 100-ft. lattice girder, 40%; for a 150-ft. pin bridge, 41%; and for a 250-ft. pin bridge, 45 per cent. As an average, 42% was assumed, and, with a live load increased by this amount, with the corresponding percentage for impact and a unit stress of 20 000 lbs. per square inch, the web members and the floor were proportioned. This, of course, is exact only when the wheel spacing and the ratio of the wheel loads would remain the same.

The resulting increase in sections was comparatively small, and, as a rule, affected only the diagonals and counters, the floor and the connections, and in some cases necessitated the use of extra counters. For the details, unit stresses increased by one-third over their amount otherwise allowed have been used.

The Pennsylvania Railroad Specifications of 1897 contain a similar clause; it allows for 100% increase of live load and a unit stress of 30 000 lbs. per square inch, respectively, twice the unit stress otherwise allowed.

It is interesting to note that in Europe, although specifications using a percentage of the live load for impact which varies with the loaded length have been proposed (by Professor Melan, 1893), they have not been officially recognized as yet; some of the railroads using a more or less changed Launhardt-Weyrauch formula (France, Switzerland and part of Germany), others using a higher unit stress for longer

Mr. Kunz. spans than for shorter ones, but making no distinction between different members or between dead or live-load stresses (Prussia, Austria, Russia), and, again, others increasing the live-load 25 to 50%, irrespective of truss member or length of span (Prussia, Bavaria, Saxony). The most elaborate instructions concerning the proportioning of diagonals have been given in a circular of July 21st, 1899, by the Prussian Minister of Public Works:

First, where diagonals capable of tension only are used, provide counters in every panel in which a live-load increased by 50% reverses the stress of the main diagonal, the section as well as the number of connecting rivets of all the counters to be made the same as for the counters for the middle panel or panels. The counters are to be put in place after the bridge is swung, the holes for the connecting rivets on one end to be drilled in the field in such a manner as to give an initial tension.

Second, when stiff diagonals are used the counter stresses have to be determined with the original live load (without any increase), but, in proportioning the diagonals, the stresses of which can be reversed, use throughout the counter stress of the diagonals of the middle panel or panels, assuming only one diagonal as acting.

Mr. Baker. Ira O. Baker, M. Am. Soc. C. E. (by letter).—In submitting the following, the writer has the feeling that possibly it is too elementary and too nearly self-evident to warrant its presentation in this place. However, since no treatise on bridges, which is known to the writer, even mentions the matter, and since two computers of two of the best of the smaller bridge manufacturers certainly do not know of the method, it will be briefly stated. The writer has used this method in his class-room for over twenty years.

In finding the stresses in the web members of a railroad bridge, due to a uniform load, it is apparently the custom to use the same load, whatever the proportion of the span covered, regardless of the fact that the equivalent uniform-load corresponding to the ordinary train increases as the length of load considered decreases. For example, assume that it is proposed to find the web stresses in a six-panel, 120-ft. span, due to a given engine and train. The equivalent uniform load for 120 ft., according to a standard diagram, is 4 545 lbs. per lineal foot of track; and this is the load to be used in finding the stress in the inclined end post. To produce a maximum in the tie in the panel next to the end, the load should cover four panels, or $4 \times 20 =$ 80 ft. The equivalent uniform-load for 80 ft. is 4 650 lbs. per foot, and this load should be used in finding the stress in the tie in the second panel. Similarly, the load for the third panel is 5 050 lbs. per foot, and for the fourth panel 5 670 lbs. per foot.

If the writer is correct in his belief that the above method is not generally used, then when the maximum web stresses are computed

by using an equivalent uniform-load, they are generally less than the actual stresses, and the difference is greater in the counterstresses than in the main stresses. Mr. Baker.

HENRY S. PRICHARD, M. Am. Soc. C. E. (by letter).—The writer does not share Mr. Seaman's belief that an intimate connection exists between insufficient provision for counterstresses and the use of wheel concentrations in designing bridges. At any rate there is nothing to prevent those who prefer the use of wheel concentration from having sound judgment regarding the proper provision for counterstresses. It may be that the present loads will not be greatly increased in the future, but the author cannot agree with Mr. Seaman that there seems to be no possibility of doubling the loads for which the heaviest bridges are now designed. At any time during the past 15 years there have been those who thought that the increase in loading had reached its limit. If the limit has now certainly been reached, then the provision proposed for counterstresses would probably be greater than necessary, but so also would be the provision generally made for all the other stresses. Mr. Prichard.

As Professor Jacoby points out, an increase of 100% in the live-load stresses, besides the addition for impact, is not simply to provide for the gradual future increase of loads. It is in reality a factor of safety against any contingency in which the effect of the load on counterstresses is greater than would be anticipated from the normal loading. The author would not consider it wise to make this factor of safety as low as 42%, as Mr. Kunz stated was done in the cases he cites of bridges for the Missouri, Kansas and Texas Railway. If, in those bridges, the effect of the load was, from any cause, slightly more than 42% in excess of what would be anticipated from the normal load, then, as regards all stresses other than counterstresses, the effect would simply be to increase the stresses to slightly over 20 000 lbs. per square inch; but, for counterstresses, the effect might be to put counterstresses where no provision whatever has been made for them.

The writer prefers the provision cited by Mr. Kunz as given in the Pennsylvania Railroad Specifications of 1895. This provision, which consists of doubling both the live load and the allowed unit stress, was made at the writer's suggestion, after a preliminary draft of the specifications had been submitted to him for criticism.

Mr. Kunz's synopsis of European practice is interesting, but the instructions by the Prussian Minister, to which he refers, seem to be more complicated than scientific.

The influence of the wind in reversing stresses should be considered, as stated by Professor Jacoby, but, as the maximum wind pressure, in combination with the other conditions which tend to reverse stresses, does not occur frequently, it would seem to be justifiable, in providing for its influence, to use high unit stresses.

Mr. Prichard. Professor Baker has shown conclusively that a bridge designed for a uniform load will have less provision for web stresses in general and counterstresses in particular, in comparison with the provision for chord stresses, than it would have were it designed for the usual engine and train load. While having no connection with the subject under discussion, it may be remarked, incidentally, that Professor Baker's analysis also indicates the difficult nature of the problem which confronts those who are striving to find some simple substitute for the method of determining stresses from the engine wheel concentrations. Any one who is expert in the use of the moment diagram will find that for ordinary cases it takes less time and trouble to use the method of wheel concentrations than it would to use an equivalent uniform load if a different equivalent had to be determined for each different position of the load.

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THE REACTION BREAKWATER AS APPLIED TO THE IMPROVEMENT OF OCEAN BARS.

Discussion.*

By Messrs. ELMER L. CORTHELL, HENRY L. MARINDIN, GEORGE Y. WISNER, THOMAS W. SYMONS, H. C. RIPLEY, THOMAS D. PITTS and LEWIS M. HAUPT.

Mr. Corthell.

E. L. CORTHELL, M. Am. Soc. C. E. (by letter).—The writer hesitates to discuss this paper for the reason that it deals with an uncompleted work, which is so exceptional, as to the theory upon which it is based, that the paper describing it seems to be prematurely issued and unsatisfactory for discussion.

The writer desired sincerely that the means should be furnished to complete the work according to the plans of its designers, in order that it might be demonstrated whether the peculiar theory upon which they are based is correct; for the cost will be only about one-third of the usual cost of obtaining deep water over sea bars.

The author, however, has had the boldness to attack the methods generally adopted hitherto, and he has advanced certain theories and made certain claims for them, and in so doing has necessarily criticised plans based on other and opposed theories. It is, therefore, necessary to consider the paper carefully, which, however, the present

*This discussion (of the paper by Lewis M. Haupt, M. Am. Soc. C. E., printed in *Proceedings* for September, 1899) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to December 22d, 1899, will be published in the next volume of *Transactions*.

Mr. Corthell. engagements of the writer do not permit him to do; he can only hastily give a *résumé* of the subject.

The terms used and the words coined by the author are not entirely understood by the writer, and he may not be correct in translating them into common language.

It is not quite clear what the word "reaction" means, or such an expression as this, "the curved form of the breakwater is such as to develop the potential energy of the ebb currents, rendering them kinetic." It is easily understood that a current impinging against the concave bank of a river scours out the bed near the bank, but just how this can exist in the case of a convex bank is not so easy to understand; for it must be remembered that one of the essential features of this work is its "S"-form—a "compound and reverse curve."

It might be expected in a river that a current passing from one concave bank to another concave bank, as is the case at the Aransas channel, would necessarily make what river men call a "crossing," and that on this crossing, or reversion point of the current, there would be a slackening of the velocity and a deposit of sediment. There seems to be such action going on at Aransas and a shoaling at this reversion, which no doubt will always exist and will give a limited depth at that point unless it is deepened artificially.

The entirely new idea that "it is not velocity * * * that constitutes the working force to produce scour, but reaction" is so entirely opposed to all accepted ideas of river and harbor engineers and their experience, that it is difficult to understand.

The writer, in the course of the last 25 years, has had occasion, not only to build works, but to examine many personally, not only in this, but in other countries, as well as to conduct an extensive correspondence with engineers engaged on similar works, and he must say that the views of the author are not, he believes, in accord with successful practice. The writer will not say that a single-legged, worm jetty would not, in some situations, obtain the desired results, but to adopt it as a general principle is, he believes, unsound.

The author refers, for confirmation of his views, to the Dublin harbor works, which the writer has examined, but has found so entirely different in conditions and plans from those at Aransas that he does not see the force of the comparison. The tidal opening at the land end of the North Wall at Dublin is only 600 ft. long, and the dike itself is 9000 ft. long. The tides in Dublin Bay not only rise 13 ft. as against a little over 1 ft. at Aransas, but this great range of tide—which, by the way, is semi-daily, instead of daily, as at Aransas—and the local configuration of the shores cause peculiar and opposing currents, which this opening and the direction and extent of the "North Wall" took advantage of; and—which constitutes the greatest difference—there is

Mr. Corthell.

a solid wall 16 000 ft. long on the opposite side of the channel connected with the main land, all above water.

It is to be noted, further, that the channel, formed so successfully by scouring action through the bar, is at the narrowest place between the two jetties, and that where the "North Wall" flares away from the "South Wall" and leaves a very wide opening, the entire distance has to be dredged continually to make and maintain the depth. In fact, from the bridge at Carlisle Street, in the port, to within 1 000 ft. of the sea end of the "South Wall," which is the guiding and longest jetty, the improvement in the channel is due solely to dredging, the effect of the scour being inappreciable.

One purpose of the Aransas breakwater is to "arrest the littoral drift," and for this purpose it is built on the "windward" side of the channel. The author presents no data to show that the "windward" side is the north and east side. The writer's recollection is that the preponderance of winds is from the south and west, and that the winds and the currents are up the coast during most of the year. However, that is a matter of fact to be ascertained from the anemometer records at Aransas or Galveston. As to currents in the Gulf of Mexico, the writer believes the following to be generally the conditions:

The current enters through the Yucatan Channel, and goes out through the Florida Channel—so far, at least, as surface currents are concerned. The former moves up the coast, hugging it in the Bay of Campeachy, past the mouth of the Coatzacoalcos River at the Isthmus of Tehuantepec, past Vera Cruz, and then begins to spread out into the great area of the Gulf. Nearing the mouth of the Mississippi, but still some distance off, it gathers itself together for its passage through the Florida Straits, its velocity gradually becoming greater. All along this part of the coast there is a reactionary or eddy current which moves in the opposite direction, and which is felt as far west as Galveston and the mouth of the Brazos, though with much diminished force. At Aransas there is no appearance of the Gulf Stream for probably 150 miles off shore; it is opposite the widest expansion and the slackest current of the Gulf Stream. The winds cause the currents, and but little reliance can be placed on the westward drift. For this reason the opening which is left between the jetty and the land is of only theoretical advantage, and the writer believes it to be a positive, practical disadvantage. This was his contention in reference to the old plans at Galveston, and, as a lesson from these works, the following is abstracted from his paper on the South Pass Jetties:*

The flood volume of the river, and the full volume of the tides, are the powers which the engineer should call to his aid. Lateral outlets near the land should not be provided. Ostensibly, they are for the

* *Transactions*, Am. Soc. C. E., Vol. xiii, p. 326.

Mr. Corthell. purpose of letting the tides into the bays, but it is through them at the ebb that they seek to find the lower level of the sea, instead of taking the longer and more difficult course along the inside of the works, where there is a pronounced frictional resistance to current movement.

This important feature of harbor improvement was explained so clearly by the late James B. Eads, M. Am. Soc. C. E., in his discussion of the writer's paper that it will be useful to refer to it in order to set ourselves right on this point.

The writer has made a special study of most of the river mouths and harbor entrances on the North, Baltic and Zuyder Seas—there are over 25 of them—and he knows of no instance where there are not two jetties contracting the channel-way and concentrating the fluvial or tidal flow. These parallel jetties are not always equal in length, but there are always two of them; and while in the nearly tideless seas of the Baltic and the Zuyder Zee dredging has to be done to some extent, there is no doubt whatever that the two jetties are the cause of the successful maintenance of the deep channels which have been maintained there for many years, the depths of which are gradually increased to accommodate the increasing drafts of vessels.

Further, before quoting Mr. Eads' discussion, the writer desires to state some facts, in reference to the Tampico harbor works, which may have a bearing on the "reaction" theory of the author. They will at least furnish a remarkable proof of the correctness of Mr. Eads' views, expressed seven years before their occurrence. The details of the history of the Tampico works will be found in the *Proceedings* of the Institution of Civil Engineers.*

The Pánuco River, which flows into the Gulf of Mexico at Tampico, is one of the largest in Mexico. The area of its water-shed is about 36 000 square miles. The periodicity of its floods is very variable. Sometimes there is a period of 3 to 5 years between them. The works at the mouth of the river consist of two parallel rectilinear jetties, 1 000 ft. between center lines. They are built of rough stone, with a foundation and hearting of mattress work. The depth on the bar about one mile from the shore was about 8 ft.

The works were pushed to completion with great rapidity. They were completed in December, 1892. Two or three small rises, and the tidal flow had deepened the channel to about 17 ft.

In July, 1893, a heavy rise came. It found its usual outlet partially dammed—walls on the sides and a bar between. It raised the surface of the water at least 2 ft. at Tampico, 7 miles from the Gulf; but the force brought to play upon the bar was tremendous. Slope observations made during the rise—which lasted from July 12th to August 1st—showed that the fall of the surface at the sea end of

* *Minutes of Proceedings*, Inst. C. E., Vol. cxxv, 1895-96, pp. 243-262.

the jetties varied from about 0.5 to 2.75 ft. per mile. The velocity was over 8 miles per hour. The total amount of material scoured out of the bar and discharged into the sea in that flood was 1 201 985 cu. yds., in addition to about 7 500 000 cu. yds. of suspended matter. Mr. Corthell.

The "reaction" upon this bar by these forces consisted simply in moving a great volume of water down a steep slope; it acted vigorously upon the bed of the stream and swept it out to sea.

Another great flood, in 1895, accomplished the same results, but it, finding a deep channel, spent its force in widening and deepening it the entire distance between Tampico and the mouth of the river, and restoring the sections which had become contracted there by the deposit of sediment in the floods which had occurred during the construction of the works.

With these facts—and similar ones from other works might be mentioned—let the reasons for such satisfactory results, given in discussing the writer's paper on the South Pass Jetties and the Galveston harbor plans, be stated by Mr. Eads:*

"The Jetty System Explained.

"It is known to all engineers familiar with harbor improvements, that the jetty system is a method of deepening and maintaining a channel across a shoal by such works as shall compel the water flowing over the shoal to pass through a narrower channel; but, as this paper will be read by some who are not familiar with harbor improvements, I will be pardoned for stating the following elementary principles which control such problems:

"*First.*—A current is caused by the fall of water from a higher to a lower level, which fall is indicated by the slope or inclination of the surface of the water.

"*Second.*—Friction of the bed over which the water flows is the chief force opposed to the current.

"*Third.*—The velocity of the current will be increased by either increasing the slope of surface or by increasing the volume, or by lessening the friction.

"*Fourth.*—The friction increases as the width of the bed increases; that is to say, if the bed of the channel be twice as wide, the friction will be doubled; if its width be reduced one-half, the friction will be reduced one-half.

"*Fifth.*—The power of water to transport sand increases with the square of the velocity.

"As every grain of sand that lies upon the shoal is brought by the current and left there, because it could take it no further, it follows that we cannot hope to deepen the channel, except by increasing the current. This can be done in three ways, but in this case it is impossible to do it *at first*, except by increasing the slope of surface. After the current is *increased*, deepening commences, and as the contracted channel enlarges, the slope is gradually reduced to its former inclination. When this occurs the old rate of current would return, but the other two elements, namely, *increase of volume* and *diminished friction*,

* *Transactions*, Am. Soc. C. E., Vol. xv, p. 284, *et seq.*

Mr. Corthell. then continue the abnormal acceleration. As the deepening goes on a *reduction of slope* follows, and this finally reduces the velocity so much that deepening ceases. The slope results from the difference of the heights of the water in the Bay and the Gulf. *To facilitate* the flow from one to the other, by *leaving openings through the jetties*, tends to keep these heights nearer alike and the resulting slope between them must then *be less*, not greater. This is precisely what we do not want *at first*. * * * * * * * *

"Outlets Through the Jetties.

"As the reader may not at once perceive the mistake of leaving these openings to facilitate the flow into the bay, I will explain that, in a state of nature, the immense frictional surface over which the flood water has to pass in filling the bay retards it so much, that the tidal rise in the Bay at Galveston is probably 10% less than it is outside of the bar. On the other hand, the discharge of the ebb is retarded by the same cause, making the total rise and fall just inside of the harbor probably 20% less than it is at the same depth outside of the bar.

"The bottom of the channel is composed of separate grains of sand, without the slightest cohesion or bond. They were brought here by currents and left, because those currents, from some cause or other, became too feeble to carry them further. Now, the reader must remember, first, that a current depends upon *the slope of surface* of the water (not upon the slope of the bottom), and second, that the excavating or transporting power of the water depends upon the *velocity of the current*. This excavating power or *velocity of the current must be made greater* than it now is over the bar, or the channel cannot be permanently deepened by any artificial structures known to the science of engineering. And yet no engineer can increase this velocity *permanently*, as will soon appear. But the velocity *must*, nevertheless, be increased, or these masses of sand composing the bar cannot be *permanently* removed. How can the velocity of a current be increased? In three ways. First, by increasing the *slope of surface*, which is the cause (or rather the exemplar of the cause) of the current. Second, by lessening the frictional resistance of the bed. Third, by increasing the volume or mass of the water. We cannot avail of these last two methods *at first*, but after we shall have *once increased* the velocity by permanent works, these two last means will then ultimately come to our aid. *At first* we have absolutely no power to increase the velocity of current but by *increasing the slope of surface*. These slopes are alternately towards the sea and from the sea, accordingly as the tide fills and empties the bay. They are by no means steep, as the average maximum tides are but a little over a foot high. The force of the present currents resulting from these low slopes cannot overcome the force of gravity in the grains of sand composing the bed of the channel. With the present slopes, volume and frictional resistance, the equilibrium between the *velocity* or transporting force of the current and the force of gravity in the sand is reached when the depth of 13 ft. occurs on the bar. If we should suddenly deepen the channel by dredging, it would be larger than Nature demands under the present conditions. The bay would then fill more easily, the slopes would be lower, the velocity less, and soon after the dredging ceased, the enfeebled current would drop its burden of sand in the enlargement, and restore the present depth. On the other hand, if we could suddenly drop a million tons of sand in the channel, and reduce the depth a few feet, what would

occur? The bay would not fill so rapidly, and steeper slopes would occur in consequence. These would create greater current velocities, and the removal of the obstruction would be effected in a little while; and, with the recovery of the 13 ft., the bay would fill more easily, the slopes would diminish, and the former velocities of current would be restored. The slope results from the difference of level between the surfaces of the Gulf and the Bay. Any one can see that the more freely the water can flow from one to the other, the less difference will there be between the heights of their respective surfaces, and, consequently, the less will be the slope of surface through the channel between them, and the less will be the current velocity. This is certainly the very opposite of what *we must have* to produce deepening. We must have more rapid currents, and nothing but steeper slopes will produce them. Hence it is evident that *large openings*, and a freer communication with the bay through them, and over low, submerged jetties, are precisely what we do not want. Mr. Corthell.

"Let us suppose that jetties were, at mean tide, suddenly placed across the bar, 3 000 ft. apart, and extended from 30 ft. depth inside to 30 ft. depth outside, and were so high and tight that all the water which passes in or out of the bay is compelled to pass through them. When the tide rises it will be impossible for the bay to fill as fast as before. The tide outside rising only to its normal height will produce a steeper slope through the jetty channel than existed in that locality before, because the surface of the bay must be lower, as it will have been deprived of a large part of its normal volume of the flood. This steeper slope will produce a more rapid current into the bay, over sands which are only quiescent under the present velocity. If this slope increases that velocity from 3 to 4 miles per hour, which is not an unreasonable supposition, the transporting power of the water will be increased from 9 units to 16 units. Wherever through this jetty channel the depth is least, there will the current be most rapid. Hence the inflow through it will have the effect of cutting down the high parts of the bottom first, and transporting these sands into the deeper places. In a few hours this whole process is reversed, and a steeper slope than before will exist in the opposite direction, and the sands of this contracted channel will be swept out to the sea, and will there be distributed by littoral currents or sea waves, out of the way of the outflow from the jetties. Of course, the greatest increase of slope will only occur at first, and at the highest and lowest periods of the tide; but *as the channel deepens the volume increases, and the ratio of friction to volume decreases*, hence lower slopes will then produce equally rapid currents. Therefore the bay will ultimately fill more and more quickly, and these abnormal currents will be thus toned down until the deepening ceases and a new condition of stable equilibrium, or what engineers call "a new regimen," is established between the maximum velocity of the current and the force of gravity in the sand. When this is reached it will be found that the tidal movement in the harbor conforms more nearly to the rise and fall in the Gulf, both in height and time, and that the slopes are *lower* than at present. Because the deepening of the channel will continue until the bay will again receive all of the water due to the difference in the tidal movement, and to the normal slopes of surface occurring between the alternating levels of bay and sea through this deep channel, and as the friction which now retards the entry and exit of the water will be greatly reduced through it, a correspondingly increased quantity of water will enter the bay.

"Any excessive increase of tide caused by winds will have the effect

Mr. Corthell. of permanently increasing the depth of this jetty channel, and of making it more capable of discharging such excessive quantities of water again. By such abnormal causes the channel will be so deepened that the currents from ordinary tides will not be sufficient to disturb its bottom. The greatest work of excavation is done when the velocities are greatest. The channel with high jetties extending out to deep water would get no accessions of sand, and the depth would be permanent. It must be apparent, from this explanation, that the object sought by these officers, namely, *facilitating the inflow of the tide* into the Bays at Galveston, Charleston, Sabine Pass, etc., by leaving openings through the jetties near the shore, is precisely the way *not to do* what they want to accomplish. Besides losing through them the most important agent for deepening the channel—the ebb tide—they will admit into the channel all of the sand which the sea waves can bring. *Submerged jetties* simply aggravate the evils caused by these openings. They really extend the openings over the entire length of the jetty. They prevent, by increased friction, the rapid inflow to fill the bay; they permit the loss of the ebb tide, and thus lessen the force that should be applied to deepen the channel; and they allow the waves to transport sand over them into the channel. Openings and submerged jetties are therefore an inexcusable violation of the very essence of the jetty system."

There are some other serious objections to the single-jetty plan at Aransas. There is no concentration of the ebb-tide flow to make and maintain a deep and wide channel. There may be a narrow, curved channel along the concave side of the work, but between that and the outlet at Mustang Island there is always likely to be a troublesome shoal due to the reversion point of the current previously referred to. And beyond, at the sea end of the curved jetty, there is always likely to be a shoal bar for want of a strong issuing current to sweep it away. Again, the channel throughout its whole length along the curved breakwater is exposed to the sudden and serious influx of sand at any time by southerly storms.

In the case of the South Pass and Tampico Jetties, the bars in the sea are swept by currents passing across the ends of the jetties. The winds and storms from the southeast at the former, and the "Northers" at the latter, raise the surface of the water in the angle formed by the East and North Jetties and the shore line and produce a strong current around the windward jetty and across the fore shore of the sea bar, wearing it away and thus retarding its growth. There is no such force to rely upon at Aransas, for the wide opening between the land and the breakwater will prevent it.

This discussion is not upon a subject unfamiliar to the writer. He was the chief engineer of the project for two years during its inception, had charge of the matter when the charter was obtained from Congress in 1890, and made plans and estimates of the works, which consisted of two parallel, rectilinear jetties. He is confident, from his experience elsewhere and knowledge of the conditions at Aransas, that his plans, if carried out, would have given immediately a channel 26 ft. deep and at least 200 ft. wide at that depth, which could have been

maintained easily by keeping up the works, and without resort to dredging. Mr. Corthell.

It remains to call attention briefly to the closing paragraph of the author's appendix, in which, after making an erroneous statement about the effects of the recent floods at the Brazos, he says:

"Thus demonstrating the relative superiority of the principle of reaction by a single jetty, as applied at Aransas, to that of concentration by twin jetties at the Brazos River bar."

Let the writer say, first, there is no sort of comparison between the conditions at the two places. At Aransas there is simply a clean, salt-water, tidal connection between the Gulf of Mexico and a group of bays—Aransas Bay with 81½ square miles connected with Misquete, St. Charles, Copano and Corpus Christi Bays, which together have an area of 273½ square miles. At the Brazos, there is a great, turbulent, muddy river in times of flood, 1 000 miles long, flowing through a clayey country, and in floods vomiting its engorgement out upon the Gulf; with no tidal reservoir whatever, except what exists in a narrow river not over 400 ft. wide.

At Aransas, the normal depth on the bar was about 9 ft. At the Brazos, when the works were begun, there was a straggling channel of 5 ft., which had to be crossed by the East Jetty. The depth on the bar between the jetty lines was only 1.5 ft. Still, these works, though never completed or consolidated, due to financial reverses by the financiers of the project and for reasons outside of this project, deepened the bar and made a channel straight to sea, which, in 1896, had a depth of over 20 ft. at average flood tide, the plane of reference used.

The author states:

"Apropos of the severe floods in the basin of the Brazos River, Texas, in July, it was expected that the larger volume discharging with greater velocity over the bar at its mouth between parallel jetties would materially improve the depth. On the contrary, the soundings show but 15 ft. in several places."

The author was misinformed; for the writer communicated with the works and ascertained the following from the engineer and superintendent of the railroad and terminal:

"October 3d, 1899, Velasco, Texas.

"Letter just received. First part rise deepened channel immediately outside jetties to 23 ft., mean low water. Latter part shoaled same place to 15 ft. in one small place, but deepened again to 16.5 ft. within two weeks; now 18 ft. Very marked improvement in river channel from Velasco to and through jetties. Never such depth water before; no harm to jetties. Bar entirely outside jetties. Now channel deeper and wider in front of jetties than ever.

"E. D. Dorchester."

The writer also communicated with Mr. William H. Coolidge, the attorney of the Reorganization Committee, in Boston, which has general charge of the property, and the following was received from him:

Mr. Corthell. "My instructions from Velasco have been to the effect that almost without exception the flood removed bars, etc., so that there was a clear entrance drawing at least 18 ft., and with a very slight amount of work the depth would be 22 or 23 ft. With two or three very slight exceptions, the water was more than 20 ft. deep. The Government having taken charge of this property, I am told that a very few days of dredging will allow vessels to enter drawing 23 or 24 feet."

The writer's original report, made after one had been made by the Government Engineer to the effect that 10 ft. as a maximum might be maintained by jetties with a maintained channel of 6 ft. only; and that the $140 000, already expended by the Government without result, should not be supplemented by any additional amount, gave the opinion that a channel 20 ft. deep could be obtained and maintained by properly constructed jetties. He still believes this can be done, notwithstanding the unfortunate financial history of the work, the delays in construction and the inability of the company to complete the works at the time.

Mr. Marindin. Henry L. Marindin, M. Am. Soc. C. E. (by letter).—The method of improvement designed by Professor Haupt for Aransas Pass presents three prominent features, the most important of which, in the writer's estimation, is the "reaction principle" as shown in the curvature given to the breakwater.

The principle is based upon the well-known effect produced by flowing water in the concave bend of a stream where the impingement of the current causes greater depth than in any other part of the cross-section, and, indeed, greater than in the straight reach.

The frequency with which running water flows in curves would seem to amount to a law of flowing streams, and this is nowhere illustrated better than in the case where the stream is free to carve out its own bed, as in the Lower Mississippi; there the regimen lies in curvature, and it is only when this curvature becomes excessive, and offers too much resistance to the flow, that during some period of flood stage the water takes a straight course—in the line of least resistance—across the more or less narrow neck of land separating it from the reach below—making what is known there as a "cut-off;" but this remedy is as much a violation of the regimen in one direction as the excessive curvature of the bend which caused it was in the opposite direction. The stream almost immediately proceeds to form another bend.

That it is not the mere velocity of current which induces erosion of the bottom of a channel must be acknowledged by witnessing so many instances where, in one case, a stronger current fails to produce erosion of the bottom, and in the other case, with the bottom composed of—to all appearance—similar material, a weaker current maintains greater corresponding depths.

The New York Narrows is offered by Professor Haupt as an illus-

tration in point, and is an apt one so far as that a great depth of 100 ft. seems to be maintained by the comparatively low mean velocity of 2 ft. per second. There are instances where greater mean velocities fail to maintain 10% of the depth found in the Narrows. Mr. Marindin.

The explanation offered by Professor Haupt for the great depth in New York Narrows is that a resultant force is produced by the flood stream flowing up on the bottom for eleven hours out of the twelve, indicating a huge vertical eddy. That may be the true explanation, but, in that connection, the writer may be permitted to ask if he has not misunderstood the report from which the statement is taken? Seven hours out of twelve would be more in accord with what has been observed in other parts of the harbor.

The flood current has been observed to flow up on the bottom, at the mouth of the Hudson, for $2\frac{1}{2}$ hours before the ebb current had ceased to flow at the surface.

It has also been held that if both ebb and flood currents could be made to follow the same path or channel, although in opposite directions, the problem of improving the depth of outlets would be near a solution. This is seemingly the condition at New York Narrows.

The configuration of the shores at Aransas Pass seemed fitted for a "compound and reverse-curve breakwater." The position of the concave shore of Mustang Island made the convex part of the breakwater opposite to it possible, or, perhaps, better fitted than a straight jetty would have been, and, as the thalweg is found close to the shore, with ample depth, it only remained for this part of the breakwater to arrest any encroachment upon the channel from the north and east. Farther down, the reaction principle becomes operative until the deeper water is reached, beyond the bar.

It is not clear to the writer's mind wherein Professor Haupt has provided for meeting the requirement stated in his seventh specification, viz.:

"It (the breakwater) should not promote the growth of the bar seaward by the deposit of material removed from the channel on the outer slope of the bar."

If material is being moved by storm waves, some of it will find its way past the outer end of the breakwater, and any material held in suspension or rolled along by the ebb on the bottom of the channel will be deposited in front of the outlet, no matter what form shall have been given to the end of the breakwater.

The writer does not know how much or how little sediment is brought down by the ebb out of the Pass, but if any amount is being carried, and a bar tends to form in front of the present curved end of the breakwater, an extension in continuation of the curve could not be continued for any great distance, for obvious reasons; and a reverse curve would be detrimental to the reaction theory, without transferring the reaction principle over a "crossing" to another concave break-

Mr. Marindin. water located on the south side of the channel. Such a crossing means the formation of a middle ground, with decreased channel depth.

The second important feature in this improvement lies in having provided a wide gap at the shore end of the breakwater for the free passage of the flood tide, to fill the tidal reservoirs above the Pass, at a point which is not occupied by any considerable ebb stream, the strength of the ebb occupying, as it does, the concave shore of Mustang Island.

It is evident that where the working power of the ebb depends largely upon the drain of flood-tide waters, works of improvement should be so designed and constructed as to offer the least obstacle to the free access of the flood tide; under these conditions the building of continuous parallel or convergent jetties would be a mistake.

The third prominent feature is the location of the single curved breakwater on the side of the Pass from which may be expected the most violent movement of material, or that due to a littoral drift. The determination of this location should be based upon accurate and reliable information, previously obtained, as to the direction and strength of flood, the direction and volume of drifting sands or other material caused by prevailing winds and storm waves or by any littoral current. Professor Haupt disclaims having sought or secured any of these data, preferring the study of the topographic and hydrographic features shown on published charts, supplemented, as it must be, by the intuition of the engineer. This is well, if success follows it, but the writer is sure that the majority of engineers would prefer to have numerical data before them to mix with all the intuitive knowledge they may possess.

An analysis of the systems of parallel or convergent jetties and of Professor Haupt's system, as applied to Aransas Pass, shows them to be radically different. The application of the single "Reaction Breakwater" seems, from the results noted in the paper, to have been particularly successful in a very short time, notwithstanding the existence of an old rock jetty directly in its path and removed only in part. But the same system applied to South Pass or to South West Pass would prove a failure. In both of these cases the conservation of the energy of the current which obtains between their parallel shores and its maintenance as far as the bar, is necessary, not only to erode the bar, but to convey the millions of tons of solid matter, held in suspension and rolled along the bottom, far out into the deep water in front of the delta, where the rebuilding of the bar would be a very slow process, retarded by the action of any littoral current existing there. Parallel and continuous jetties are necessary to secure these results, and prevent the spilling of the currents laterally.

The same condition confronted the engineer at the mouth of the Brazos, and it may be said, parenthetically, that the present jetties in

their incomplete state have secured within their limits as great depths as are found in the river a mile or two above the mouth. Mr. Marindin.

The writer has endeavored to show in the foregoing remarks that, notwithstanding the success which has attended the single jetty, parallel jetties are justifiable and necessary under certain conditions.

George Y. Wisner, M. Am. Soc. C. E. (by letter).—The peculiar nature of the reports made by the Board of Engineers on Aransas Pass, in November, 1897, and December, 1898, together with the fact that the expenditure of large appropriations will soon be required for the improvement of other harbors where the conditions are similar to those at Aransas Pass, make the paper presented by Professor Haupt very apropos at the present time. Mr. Wisner.

The harbors on the Gulf Coast are of two distinct types, each of which requires radically different treatment in order to secure satisfactory results. The Mississippi River, the Brazos River and the Pánuco River, at Tampico, are all heavy silt-bearing streams during times of high water, and, if not controlled by tight jetties so as to maintain the full velocity of flow until discharged across the bar into the littoral current outside, a deposit of silt will occur and produce shoals instead of deepening the channel as generally expected. The strong currents of these rivers at flood stages are alone to be depended upon to produce scour in the channels, and all arrangements designed for securing tidal effect are not only useless, but tend to decrease the velocity of the silt-laden current, and thereby cause shoals by depositing silt.

At the entrances of tidal harbors, reversed conditions exist, and the erosive force required to deepen the channels must be obtained by accumulating vast quantities of water in the tidal reservoir, during the flood stages of the tide, and concentrating the discharge along the axis of the required channel lines during ebb tide.

Unfortunately for the success of many of the improvements attempted, the designers of the structures conceived the idea that works which had produced beneficial results at other harbors must necessarily be the proper remedy to apply. The result has been that millions of dollars have been expended on jetties which have been of absolutely no use, except to form breakwaters for the protection of dredged channels. One of the most serious difficulties which the designers of such works have apparently feared was that such strong tidal currents would be generated that the structures would be in constant danger of being destroyed, and, to avoid such a catastrophe, the jetties have invariably been located from 2 000 to 7 000 ft. apart.

Generally speaking, no deepening of channels has resulted from such construction, and whatever deeper channels now exist at these harbors have been secured by subsequent dredging.

At Galveston, with jetties 7 000 ft. apart, the dredged channel

Mr. Wisner. across the bar is from 500 to 1 000 ft. wide, and at Sabine Pass, where the jetties are 2 000 ft. apart, the dredged channel is only about 200 ft. wide, and shows no indication of increasing depth or width from tidal action. In spite of these repeated failures, the 1897 Board of Engineers not only endorsed the 1888 project for jetties 2 000 ft. apart at Aransas Pass, but condemned the unfinished curved jetty as worthless to the Government, and as an obstacle to future improvements, when their own published charts show beyond question that the results predicted for the enterprise would be realized at once upon the removal of the old curved jetty, which the predecessors of the Board located, unfortunately, on the wrong side of the Pass.

In the report of December, 1898, the Board recommends that the outer 1 000 ft. of the curved breakwater be removed, and that an extension be built on a line about 25° to the north of the present location, making an angle with the direction of flow which, it is safe to say, the currents would never follow. With a channel from 15 to 23 ft. deep, already obtained by the incomplete works now in place, it is hard to conceive why such structures should be removed, and be replaced by others which have been proven to be failures at other ports, where the conditions were similar, especially as the estimated cost is three times that necessary to obtain a 20-ft. channel by the completion of the works built by the Aransas Pass Harbor Company.

While it is true, as stated by the author, that parallel jetties are not well adapted for securing tidal scour at the entrance of tidal harbors, it is equally true that a single curved jetty at the mouth of a silt-bearing river would not only fail to produce a good entrance channel, but would not maintain it if obtained by dredging or other methods. The case mentioned, where the flood in the Brazos River failed to deepen the channel, is a direct proof of the principle involved, and was the result expected by those familiar with the situation.

The jetties at the mouth of the Brazos River have never been completed so as to maintain a full velocity of flow until the silt-laden water is discharged into the deep water of the littoral current outside the bar, and, therefore, in times of heavy floods a portion of the sediment is dropped at the point where the river current is slackened. The experience at the mouth of the Mississippi and Brazos Rivers indicates clearly that single jetties, no matter what their shape, would not maintain navigable channels at the entrances of those harbors.

To every careful investigator of the physical conditions at the entrances of Gulf harbors, there is plenty of evidence of the kind of remedy which should be applied to secure any reasonable improvements of the channels. The littoral current is a controlling force, which is certain to cause either success or failure, according to whether properly utilized or not.

Mr. Wisner.

At the mouths of silt-bearing streams, the suspended matter must be carried entirely from in front of the entrance before being deposited by a change in the velocity of the river current, and the shore drift must be prevented from entering the channel; two conditions which necessitate the construction of jetties on each side of the entrance, to deep water on the outside of the bar. The action of the littoral current in flowing past any system of jetties is to pile up the sand drift in the angle of the shore and jetty on the windward side, and produce an eddy on the lee side of the entrance.

On the Gulf Coast, the resultant of the littoral current is from the northeast to the southwest, and in the case of sediment-bearing rivers the eddy builds up immense shoals to the westward of the entrance, and at tidal harbors causes a reverse current along the shore to the westward of the jetties, which in connection with wave action may cause great damage by erosion.

This shore current, caused by the eddy to the westward of the jetties, has in certain instances led observers to believe that the littoral current was from the west.

At Galveston, the erosion of the city front from this cause has been so extensive that suits have been started in the Court of Claims to recover compensation for the damages caused by the construction of the jetties at the harbor entrance.

If the curved jetty designed for the improvement of Galveston Harbor, in 1883, by H. C. Ripley, M. Am. Soc. C. E., had been adopted, it would unquestionably have produced a deep channel across the bar with a saving of several million dollars in the cost of the improvement, and would have shifted the eddy caused by the littoral current far enough eastward to have caused but little erosion along the city front.

The construction of jetties at Aransas Pass will produce an eddy effect similar to that developed by the structures at the entrance of Galveston Harbor, and, if two jetties are built out to deep water, the west end of Mustang Island will be eroded, whereas, if only a single curved jetty be constructed on the north side of the channel, the center of the eddy will be shifted eastward, and the reverse current will combine with the ebb currents at the outer end of the channel. A short spur will be a necessity at the west side of the Pass to prevent the reverse current from the littoral eddy carrying sand into the channel. This spur should be built above high water as far out as the wreck *Mary*, and will tend to limit the action of the eddy and diminish erosion on the front of Mustang Island.

One of the most interesting instances of parallel jetties producing the opposite effect to that expected is that at Cumberland Sound, Ga., where the accumulation of drift between jetties 3 900 ft. apart became such that, in 1895, the channel shifted across the south jetty, and it

Mr. Wisner. became necessary to remove a portion of that jetty, to prevent the port of Fernandina from being completely closed.

It became evident, in 1890, that such a result was likely to be obtained, and O. M. Carter, M. Am. Soc. C. E., Captain, Corps of Engineers, U. S. A., the local engineer in charge of the improvement, recommended that the project for parallel jetties be abandoned, and that a single curved jetty be constructed on the north side of the entrance, to shut out the sand drift from the north and guide the tidal flow across the bar on the line best adapted for maintaining a channel with natural forces.

The deadly wisdom of the engineer board, whose consideration of a few hours outweighs the conclusions of the resident engineer who gives years of study to the conditions, proved fatal to the project.

There has been $1 787 000 expended on the work, which has in no way improved the entrance to the harbor. A continuous appropriation was made by Congress in 1896, to complete this project, of which over $1 500 000 remains unexpended.

It will be interesting to note how this vast amount of money is to be expended, especially as the engineer boards which have considered the problem at various times have practically admitted that the improvement was a failure, but did not have the courage to "advise any radical change in the general location of the jetties as originally proposed."

The conditions at Cumberland Sound are practically similar to those on the Gulf Coast, except that the rise of the tide is about three times that at Aransas Pass; and there is no reason why a single jetty would not produce as beneficial results as at the latter place, where an incomplete structure has deepened the channel from 8 to 18 ft. in a few months.

Particular attention is called to the fact that the breakwater at Aransas Pass was constructed under the plans of the consulting engineers, without causing the bar to be pushed seaward the slightest measurable amount, an achievement which has never been accomplished at any other port on the Gulf Coast.

The action of erosive and bar-forming currents has been quite fully observed by the engineers who have had the actual charge of harbor construction on the Gulf Coast, and, while the results have not been published in the official reports, many of the facts have been discussed in the *Transactions* of this Society, and it is hoped that this paper will make the record still more complete.

So far as the writer is able to judge, these observations have established quite fully the following conditions and principles pertaining to harbors on the Gulf Coast.

1. The littoral current, during a season when effective, flows from the northeast to the southwest.

2. The sand moved along the coast by the littoral current, when stopped by jetties extending into the Gulf, accumulates in the angle formed by the east jetty with the shore; and the sediment carried into the Gulf by silt-bearing rivers is carried to the westward of the harbors, forming shoals extending seaward from the entrance of the jetty channels. Mr. Wisner.

3. The littoral current flowing past the outer ends of jetties or breakwaters creates an eddy west of the entrance, which, at the mouths of sediment-bearing streams, builds up shoals in the quiet water at the center of the eddy; and at the entrances of tidal harbors causes reverse currents, which, in connection with wave action, erode the shore west of the structures.

4. A single jetty will neither make nor maintain a navigable channel at the mouth of a silt-bearing river.

5. Parallel jetties at the entrances of tidal harbors will not create navigable channels, but will generally maintain such a channel after having been obtained by dredging.

4. Where strong littoral currents exist during the season when sand movement occurs, a single curved jetty, properly designed with reference to the volume and velocity of flow through the pass, will create a navigable channel without aid from dredging, but it is preferable in all cases to aid the tidal action by dredging, not only to hasten results, but to insure a proper alignment of the channel.

Thomas W. Symons, M. Am. Soc. C. E. (by letter).—The single jetty at Aransas Pass, or "reaction breakwater," as it is called by the author, is by no means a new thing in the engineering world, and, individually, it does not mark "a distinct advance in the resources of maritime engineers in the improvement of ocean bars," as stated by Professor Haupt. In fact, officers of the Corps of Engineers, U. S. A., have designed and have been building just such structures for years, and some of the most successful works of the Corps have been accomplished by adopting in the main the principles enunciated by Professor Haupt. Mr. Symons.

The writer believes sincerely that many harbor bars can be improved by a single jetty, or breakwater, if the name be preferred, to fully as great an extent, and in some cases to a much greater extent, than by parallel or converging jetties, and at enormously less cost, provided the single jetty be properly located to take advantage of all the natural forces at work. This belief is founded, not on theories only, but on actual experience in planning, locating, building and observing such works and their operation. The writer's experience on the Pacific Coast was so convincing as to this, that, in planning works for the improvement of Grays Harbor Bar, Wash., he designed a single jetty, and, Congress having made provision therefor, it is now being constructed under the supervision of Harry Taylor, M. Am. Soc. C. E.,

Mr. Symons. Captain, Corps of Engineers, U. S. A. This jetty has not only been described in official reports, but was the subject of a paper before this Society.* This Grays Harbor jetty is, in every essential respect, such a "reaction breakwater" as described in general terms by Professor Haupt.

The single jetty at Coos Bay, Ore., located and built under the writer's direction and supervision, 1890 to 1895, is a "reaction breakwater," and operates in practically the same manner as described by the author, in determining, deepening and maintaining a channel over the bar. This single jetty was located with a view to securing the maximum results in the shortest time and at the minimum of expense, and was eminently successful; a deep bar channel was developed and has been maintained in a permanent position ever since. A second jetty was planned to be built at Coos Bay, but as the first one has accomplished about all that was hoped from the two, it will probably be deemed unnecessary to build it. The great single jetty at the mouth of the Columbia River, which has accomplished so much for the commerce of Oregon, is a "reaction breakwater." So is the single jetty at San Diego, Cal.

These jetties fulfil nearly all the conditions laid down by Professor Haupt. Their principal point of variance is that they are not constructed on "compound and reverse curves," and have no gap or opening near the shore. In the localities mentioned these curves were not at all necessary, as there was no trouble in securing a deep channel inside the outer end of the jetty at either place. The bar, in advance of the jetty, was the real difficulty, and the channel across it was to be secured by directing the tidal currents upon it in a fixed location. In the instances mentioned this is accomplished just as well with the straight jetties as it could be with curved jetties. In fact, it was necessary in almost every instance to protect the jetties against undue erosion by groins built out on the channel sides. The construction of the "breakwater," so called, at Aransas Pass, is not made clear in the paper. It is apparently a brush and stone structure, a large part of which is founded on sand in 6 to 8 ft. of water. Alongside this and close up, the channel has been eroded to 18 or 20 ft. in depth, according to the map. This condition might well excite alarm for the safety of the structure. The possible undermining may, however, be guarded against by some construction not alluded to in the paper.

Along our Northern Pacific Coast there is an average tide of 6 to 7 ft., and the storms are much more frequent and severe than in the Gulf, all of which, of course, makes the problem of bar improvement vastly different from that at Aransas Pass.

Neither was there, at any of these Northern Pacific Coast harbors, any need to leave a gap near the shore to permit the tide to enter

* *Transactions*, Am. Soc. C. E., Vol. xxxvi, p. 109.

freely. The jetties as located and built leave the door open almost to its full width for the entrance of the tide, and there is practically no choking down of the flood tide, which, theoretically, there is with parallel or converging jetties. Besides, at these Northern Pacific Coast points there is a very practical objection to leaving a gap near the shore. The rough and tempestuous character of the waters prohibits absolutely the building of the structures from floating plant, and they must be built from tramways extended out from shore. If the tramway should be built and a gap left in the brush and stone jetty built from it, it is certain that the unprotected portion of the tramway would be undermined and destroyed. Mr. Symons.

Judging from the map, Fig. 2, this cutting has commenced in the gaps left in the Aransas Pass structure, and it is quite within the bounds of possibility that this may extend, and chanels develop therefrom, to a degree which will lessen seriously the flow in the desired channel. If the flood enters freely through the gaps, the ebb will also run out freely, and it would seem the part of wisdom, where there is so little tide as at Aransas Pass, to control as much of it as possible. The sill, *G E*, Fig 1, would at least be necessary, in the writer's judgment.

In the case of Grays Harbor, the natural bar channel had an apparently unswerving tendency to move from north to south, keeping on with this movment until its extreme southerly position was reached, then breaking out again to the north and taking up its slow movement to the south. The jetty was located to interpose itself against this southerly movement with the hope and belief that the pressure to the south would continue, and that this pressure on one side opposed by the jetty on the other, and the inflowing and outflowing tidal currents, would produce the necessary agitating and scouring current-effects necessary to determine and maintain a good deep channel and direct it permanently over the bar in the selected location. This work was planned in 1894-95, and is truly a "reaction breakwater" in all essential features, as described and named by Professor Haupt. Whether it will fulfil all that the writer hopes for it remains to be seen, and can only be determined in the future.

The great Columbia River jetty, planned in 1882, is also such a "reaction breakwater," interposed against the tendency of the bar channel to move to the southward.

At Coos Bay the movement of the bar channel was to the northward, and, consequently, the jetty was interposed against it on the north, and this northerly tendency and the reaction of the jetty have determined and maintained an excellent, deep-bar channel, permanent in location, ever since the jetty was carried out sufficiently far to make its influence felt. The writer is unable to believe that, in the great majority of cases, the curved trace, indicated by Professor Haupt as

Mr. Symons. essential, is at all necessary, or the best. There may be cases, however, where such a trace best lends itself to the situation and control of the forces at work. Each particular case must be judged by itself.

While the writer is a believer in the single-jetty system, wherever applicable, he also believes that there are cases where one jetty would not be sufficient, and where parallel or converging jetties are necessary. A safe rule to adopt, and one by which the writer was governed in designing all the later works of harbor bar improvements which he has had in charge on the Pacific Coast, is to locate the first jetty to be built as if it were the only one to be built, and hope for the best. Under this plan, if it be found that the results obtained are not satisfactory, then other works, a second jetty, perhaps, can be added to better control the natural forces and direct the flowing waters. The writer's experience causes him to be very chary of predicting absolutely the results which can be obtained from any structure built to control the waters and sands of an ocean bar.

Professor Haupt and all concerned are to be congratulated on the results obtained at Aransas Pass. The very marked improvement in such a short time mentioned, however, naturally causes one to ask whether there is a certainty that it is permanent. This is mentioned because, in conducting harbor works on the Pacific Coast, very extraordinary results were obtained at times, but they were not permanent, and the real value of the works could only be determined by finding out the minimum depth and width of channel which could be depended upon under all circumstances. The author recognizes this uncertainty by his statement that "modifications, however, are always to be expected in depths due to variations in the natural forces." These modifications may in time, and probably will, render the entire or partial closing of the gap in the jetty, and the construction of groins for its protection advisable and necessary.

If the author had extended his studies to the works of harbor improvement designed and carried on by the Corps of Engineers, U. S. A., on the Pacific Coast, his claim as to the epoch-making character of the Aransas Pass structure would probably have been greatly modified, in view of the fact that structures practically identical in character were designed, located and built years before it was thought of.

The correct principles of the single-jetty system appear to have been applied very successfully at Aransas Pass, and Professor Haupt has invented a new name for such a structure. The writer cannot, however, believe that the engineering world will accept the name "breakwater" as properly applicable to the structure at Aransas Pass. It is not designed to and does not perform the functions of a breakwater as ordinarily understood and defined. It is a training wall or directrix to the flowing waters, and the name jetty fits it much better than the

name breakwater, and is more in accord with good usage and common understanding. A breakwater is from its very nature a reactionary structure, its object being to deaden the force of the waves, which roll in from ocean or lake, by interposing its weight and bulk in their path, reacting against them. To speak of a "reaction breakwater" is a redundancy. Mr. Symons.

H. C. Ripley, M. Am. Soc. C. E. (by letter).—The adoption of the reaction or curved breakwater, for the improvement of a harbor entrance, is without doubt the greatest advance made in the science of river and harbor engineering that has ever been recorded in this country. For this reason this paper will be read with especial interest by those engaged in this branch of the profession. Mr. Ripley.

The principles herein applied, however, are as universal and well established as the law of gravitation itself.

These principles are:

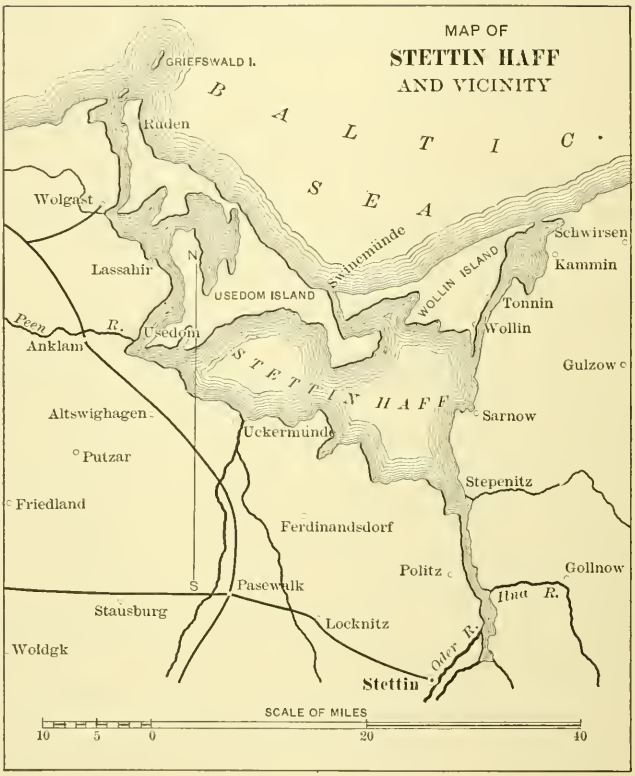
First.—That of reaction, by means of which, water, constrained to flow in a curve, excavates and maintains a greater cross-sectional area of channel than when flowing in a straight course, and the ratio of depth to section is always greater in a curved than in a straight channel, under otherwise similar conditions. For examples of these effects, see *Transactions*, Am. Soc. C. E., Vol. xxiii, page 153, where it is shown that for the Brazos River the average sectional area is 13½% greater in bends than in reaches, and the average maximum depth is 58% greater in bends than in reaches.

Second.—That of centrifugal force, by which the particles of flowing water tend to hug the concave side of a channel. This principle makes it possible, by means of a single curved structure, to concentrate the ebbing waters of a tidal harbor or the discharging water of a river into a single channel of limited width, and convey it across the bar as effectively as can be done by means of two parallel jetties, where the aggregate length is double that of the single curved structure.

Numerous examples of natural concentration of flowing water into channels of limited widths by this principle are to be found in the bays and sounds along the Atlantic and Gulf Coasts. A noted example is that of the channel of Galveston Harbor along the city front. This channel has a width of less than 1 000 ft. between the 6-ft. contours and a maximum depth of over 30 ft. This concentration of the tidal flow is due to the curved form of the shore line of Galveston Island, which brings into action the principle of centrifugal force.

These two are the cardinal principles involved in the application of the reaction or curved breakwater to the improvement of a harbor entrance. The writer thinks that no one will question their universality. Nor are these principles of recent discovery. They were recognized

Mr. Ripley.



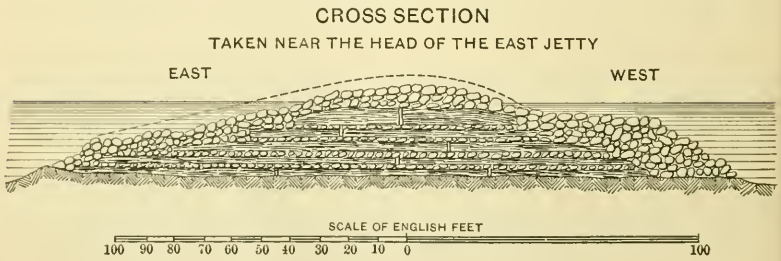


FIG. 3.

Mr. Ripley.

PLAN SHOWING
JETTIES OF THE HARBOR
OF SWINEMUENDE

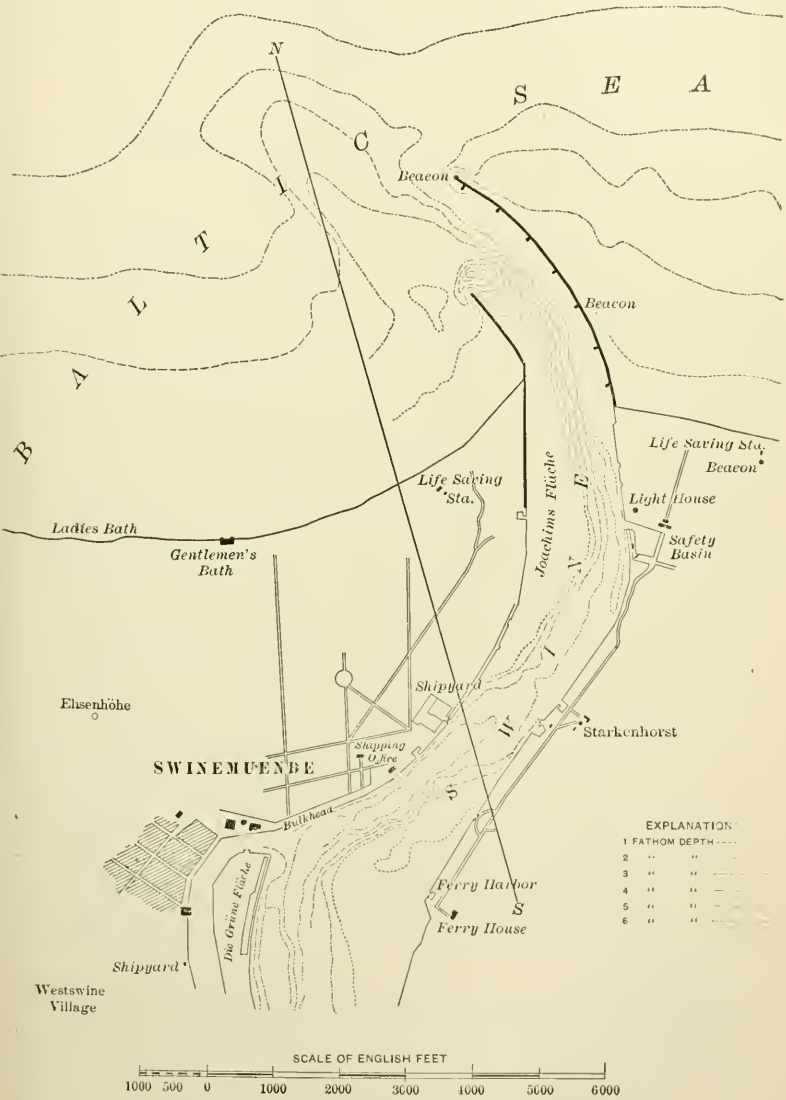


FIG. 4.

Mr. Ripley. more than three-quarters of a century ago, by engineers having to do with harbor works.

The improvement of the entrance to the harbor of Swinemunde, Prussia, on the Baltic Sea, is an example of the successful application of these principles by means of a curved jetty extending from the shore on the east side of the entrance, across the bar, a distance of 3 900 ft., with a somewhat shorter angular jetty upon the west side of the entrance.

General Gillmore, in describing this work in 1881, wrote:*

"Several harbors on the Baltic Sea have been improved by means of jetties.

"Perhaps the most successful example of this method of improvement is shown at the entrance to the harbor of Swinemunde, the outport of the City of Stettin, an important commercial town of Prussia. * * *

"The west pier or jetty was designed to arrest the progress of the sand brought to the entrance by the littoral current, besides confining the currents of the outflowing waters. * * *

"The well-known fact that near concave banks, the currents of flowing water generally create and maintain a deep channel, induced the designer of the Swinemunde jetties to construct them on a curved plan, to secure ample depths, at least, on one side of the entrance channel. * * *

"The curved plan proved entirely successful, inasmuch as considerable depths were produced next to the east jetty; in fact, the opinion has been expressed that a more gentle curvature might have been better, as the depths in the concave bend are actually greater than needed, while on the opposite side, extensive shoals are found, necessarily reducing the width of the navigable channel and requiring the frequent use of dredging machines. * * *

"As a general result of this improvement, it is reported that the principal channel, in which the depths were only 7 to 7½ ft. before the works were begun, now has a navigable depth of more than 20 ft., which has been maintained for nearly half a century."

The plan of this work is shown in Figs. 3 and 4. While it is stated that the "curved plan proved entirely successful," the work as a whole has not been so, from the fact that the curved jetty was placed upon the lee side of the littoral drift, which encouraged its accumulation on the channel side of the curved jetty, with a resulting growth of the bar seaward, which will necessitate the ultimate extension of the works to prevent its overlapping the outer end of the jetty. Had the curved jetty been placed upon the windward side, the littoral drift would have accumulated next to the shore and its seaward growth would have been much slower.

About 16 years ago, the writer designed a detached curved breakwater for the improvement of the entrance to Galveston Harbor. Its location is shown by the heavy broken line in Fig. 5. The line of the North and South jetties as constructed is also shown. The condition

* Report, Chief of Engineers, U. S. A., for 1881, p. 1060.

Mr. Ripley.

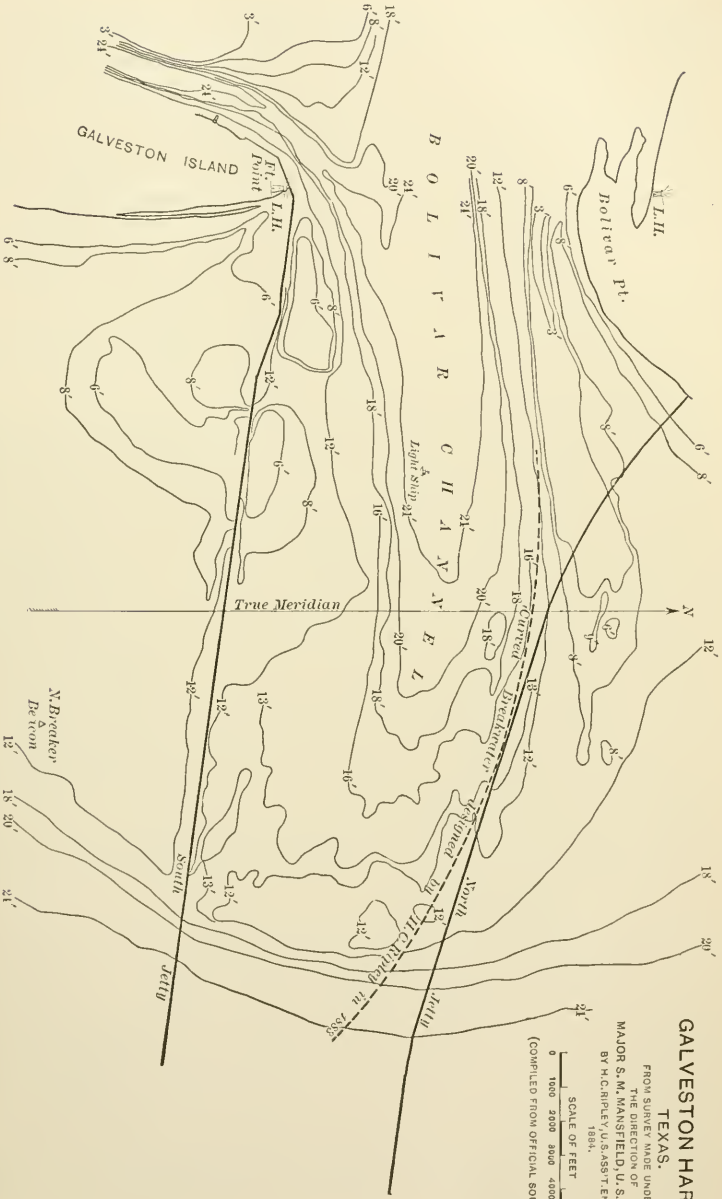


Fig. 5.

Mr. Ripley. of the bar and channels is that of the time of the survey in 1884. The present position of the shoalest part of the bar is outside of the line joining the outer ends of the jetties. At this point, the resultant littoral movement is to the southwest, as is indicated by the gentle curve of Bolivar Channel to the south as it approaches the bar.

The breakwater is placed on the windward side of the channel, and its curvature and position conform to the curvature and alignment of the natural channel, and as a resultant of all the forces acting upon it. The discharging water from the tidal basin meets this curve tangentially, and, upon the principle of centrifugal force, will be constrained to follow the curve throughout its length. The channel will therefore have no greater width than at the point of meeting the curve, but, on account of its curved form, somewhat greater depths will be developed and maintained than would be the case if the same movement of water were confined between parallel straight walls.

Its position on the windward side of the channel is such as to intercept the littoral drift and protect the channel from its degrading effects; and the littoral movement, being normal to the breakwater at its outer end, will oppose no dynamic resistance to the ebb discharge. The existence of this breakwater would exert the least opposition to the admission of flood water to the tidal basin consistent with the control of the ebb discharge, and it is probable that, with the development of the channel across the bar, an actual increase in the tidal prism would result.

This design was approved by the United States Engineer Officer then in charge of the work, and its adoption recommended. The results at Aransas Pass have demonstrated fully that, had its construction been authorized, there would have been saved to the Government, in the improvement of this one harbor, not less than $4 000 000 in money and about four years in time.

There are many harbors on the Atlantic, Pacific and Gulf Coasts needing improvement, where the application of the reaction breakwater would be entirely feasible.

The curved jetty, involving the same cardinal principles as the reaction breakwater, would also be applicable at the mouths of many rivers, to the great saving in the cost of improvement.

After much study of the local conditions at the mouth of the Coatzacoalcos River, Mexico, the writer has proposed a single curved jetty for the improvement of the entrance to that river, with the estimated saving of about $1 000 000 in gold as compared with the estimated cost of two parallel jetties.

With the results at Aransas Pass, the writer is entirely confident that the improvement of the entrance to the Coatzacoalcos River would be entirely successful with the construction of the single curved jetty proposed.

THOMAS D. PITTS, Jun. Am. Soc. C. E.—The speaker had the good fortune to be engaged as Assistant Engineer on the works at Aransas Pass for over two years, and, naturally, he has been much interested in the results. Mr. Pitts.

From the depths maintained in the gorge, it would seem that the ebb-current velocity and tidal volume are sufficient to maintain a depth of 20 ft. or more on the bar, if properly controlled and directed. The speaker believes the results so far obtained show that the curved jetty on the north side of the channel, which was designed by the author, would be amply sufficient to control and direct the current, if completed.

In reports made to the Chief of Engineers, U. S. A., regarding contemplated improvements at Aransas Pass, it has been said that it would be very difficult to build and maintain a jetty there on account of the quicksand foundation. The curved jetty, built by the Aransas Pass Harbor Company, is founded on mattresses made of fascines of live oak and bay brush, the fascines being about 9 ins. in diameter and of varying length, according to the width of foundation desired. The mattress sinks into the sand somewhat when first laid and compresses considerably when fully ballasted, but it seems to last for an indefinite time. The foundation of the south jetty, built by the Government about 1885, was a similar mattress, and during Colonel Goodyear's blasting operations on the site of that jetty fragments of the mattress arose to the surface in considerable quantity. The wood of these fragments was quite sound, but the bark was partially decayed. A portion of the north jetty has now been in place for over 4 years, and no work of any kind has been done for more than 3 years. During that time there has been no evidence of any settlement of the foundation, the small amount of subsidence noticed being amply accounted for by the consolidation of the stone in the body of the jetty, and by loss from the top due to heavy seas. In this last respect, the speaker has been surprised that the jetty has stood so well, in view of its exposed position, and the fact that no repairs whatever have been made.

Between August 1st and December 31st, 1895, a portion of the jetty 1 250 ft. long, B—C (Fig. 1), was completed to full size and section; a mattress was laid from B to F, 1 000 ft., and from C to D, 1 500 ft. The channel depths increased rapidly until, in November and December, a channel of 13 ft. could be traced across the bar. At about that time the foundation of the old Government jetty was uncovered, and the increase in depth ceased. Although the removal of a portion of the old foundation was urged repeatedly by the Consulting and Chief Engineers, no funds were provided for that purpose; but during the spring of 1896 the portion C—D of the jetty was raised to the level of high water, and the foundation was laid for a distance of 1 400 ft. from D toward E.

Mr. Pitts. During August and September of the same year the mattress was extended to *E*, and the jetty was raised to low-tide level between *D* and *E;* but it was built of much less than full section, and two gaps of a total length of about 500 ft. were left. Until this last work was done between *D* and *E* the ebb discharge was practically not controlled at all, most of the flow passing to the northward of *D*, and the current across the bar was scarcely appreciable.

The condition of the jetty in the spring of 1896 was such that it caught and caused to be deposited on the bar the sand carried along the shore by the northerly current which prevails at that season. This current was of unusual strength and duration that year. In consequence of this sand deposit, the depth on the crest of the bar decreased so that there was only 5½ or 6 ft. of water in September. As soon, however, as the portion *D*—*E* of the jetty was partially completed as described, the current across the bar was noticeably increased, and the channel depths began to increase slowly. This increase was helped materially by the explosion of dynamite on the bar by Colonel Goodyear; chiefly by the destruction of a part of the foundation of the old Government jetty, but also partly by the softening of the whole bar, caused by the shocks of the explosions.

The increase in depth, which began in the fall of 1896, has been continuous since; being slower, or perhaps even ceasing entirely, during the spring and summer, and much more rapid during the winter, when severe "Northers" are of frequent occurrence. At such times the current is very rapid, surface velocities as high as 6½ and 7 ft. per second having been measured by means of float observations. These were probably not maximum velocities, as the engineering force had no facilities for making current observations during the times of greatest discharge.

The speaker's latest information regarding the depths on the bar is by letter from the keeper of the Life Saving Station at the Pass. From this he learns that there has been but little change in the channel during the past summer, but that the depths on the outer slope of the bar have increased somewhat. In the absence of a complete survey, it is not safe to assume too much; but the increase in depth on the outer slope would seem to show that the jetty is serving one of the purposes for which it was designed, *i. e.*, to prevent the advance of the bar seaward.

The south jetty, built by the Harbor Company in 1892, as mentioned by the author, was carried out some distance during the summer, and was apparently successful for a time. A deep channel was formed along the north side of the jetty, and there was some increase in the depth on the bar. The improvement was lost, however, as soon as the southerly littoral currents set in, in the fall. There was no improvement the next year, and a large part of the

jetty had disappeared by the summer of 1895. This jetty was built of wooden caissons about 7 ft. in diameter, sunk in the sand and filled with sand and rock. A portion of this jetty is shown extending eastward from "Co. Wharf" on the map (Fig. 1). As will be seen, it did not in any way control the current, serving merely as a protection against the sand carried along the shore to the northward. In fact, the ebb current is deflected to the north by the revetment laid by the Government between "Co. Wharf" and Turtle Cove, and trends away from the shore line of Mustang Island before reaching the inshore end of the south or Nelson jetty. Mr. Pitts.

The curved north jetty was designed by the author, not only to protect the channel from the inflow of sand from the north, which movement the speaker considers more important than that from the south, but also to control the current and compel it to scour out its own channel. The speaker does not believe that a straight jetty would serve the same purpose in this location, as suggested by Major Symons, for he thinks such a jetty would only deflect the current, without concentrating it, and that it would then spread out, fanwise, and lose its velocity and strength. With the curved jetty, on the contrary, the change of direction is gradual and continuous, and the conditions are similar to those in the bends of a rapid river, where it is well known that the greatest velocity and depth is found next the concave bank.

Major Symons also mentions four harbors on the Pacific Coast where single, straight jetties have been used with gratifying results. The similarity of the cases does not appear to the speaker, however. In one of these instances, the mouth of the Columbia River, there is a very large fresh-water flow, and in all of them the tidal flow is many times as great as that at Aransas Pass. In all of these cases the entrance proper, between the headlands, is wide and deep, as shown by maps published in the Reports of the Chief of Engineers, U. S. A.; and there is a rise and fall of tide of 5 or 6 ft. The conditions are such that the bays doubtless fill completely at each tide, and the single jetty does not contract the entrance so as to diminish the tidal inflow. The depths and widths maintained in the gorges are far more than sufficient for any possible commercial needs; hence it is enough to control a part only of the ebb flow.

At Aransas Pass, on the other hand, the entrance is naturally comparatively narrow, and the average rise of tide is only about 0.8 ft. It is therefore necessary, not only to control as large a part of the ebb flow as possible, but also to give the most ample opportunity for the flood tide to enter. The purpose of the opening left from E to G (Fig. 1), was to give a greater width of flood tide entrance, and the speaker believes that it is of material value for that purpose. It is true that a considerable quantity of water is lost through G–E during ebb

Mr. Pitts. discharge, but here again the water is thrown against the Mustang Island shore by centrifugal force. and the loss is limited to a surface one only. This is proven to the speaker's satisfaction by the fact that since September, 1896, when the foundation was completed to *E*, and the jetty from *D* to *E* was brought to a partially effective condition, there has been no tendency to scour in this opening; in fact, the depths have decreased somewhat.

The conditions at Tampico and at the mouth of the Brazos, where improvements have been made by means of parallel jetties, are radically different from those at Aransas Pass, on account of the large fresh-water flow at both of those places.

Mention has been made of the probability that shoaling would take place at the "cross-over." There would doubtless be some shoaling at that point, but the speaker does not think that the depth there would be the governing depth. He thinks the loss of water over the shoal to the southward would cause the governing depth to appear in the concavity of the jetty, and that it would not be less than 20 ft. under any circumstances, and probably much more. The latest chart available, from a survey made last winter by the United States Coast and Geodetic Survey, shows depths in the middle of the "cross-over" of about 17 ft. The speaker believes that the present bar acts as a dam to check the ebb current and to prevent scour on the inner slope, and that with the opening of a clear waterway of 20 ft. depth across the bar, the depths in the "cross-over" would be increased materially.

The fact that a current velocity which is abundantly sufficient to transport a given material will not always scour it from its bed was well shown at Aransas Pass. The material along this coast is a round-grained sand of light weight, which remains suspended in the water for a long time when once put in motion. Yet the speaker has seen this sand stand for days at a slope of 1 on 2 without perceptible erosion, notwithstanding the fact that a current of 3 to $3\frac{1}{2}$ ft. per second was impinging on it during every ebb tide. But the slightest shock, such as the explosion of a charge of dynamite under water in the vicinity, is enough to cause such a bank to slide. Any obstruction to the free flow of the water will cause eddies which produce rapid scour over a limited area. For this reason the speaker does not think that the stone from the foundation of the old Government jetty which was scattered by the dynamite explosions prevents scour. On the contrary, if that stone is as well scattered as he believes it to be, he thinks that it is a help rather than a hindrance, on account of the local scour produced.

The average range of the tides at Aransas Pass is less than 1 ft.; it is given on the Coast Survey charts as 0.7 or 0.8 ft. But there are times when the rise and fall are much greater. A long-continued southeast wind will raise the water 3 ft. or more above mean high water, and a succession of severe "Northers" will lower it as much

below mean low tide. The speaker has seen the tide fall more than 3 ft. in less than six hours during a severe "Norther." As regards the character of the tidal curve; at the time of neap tides there is a well-marked diurnal rise and fall; but at spring tides there is but one pronounced tide in the 24 hours. At the time of spring tides, however, a short rise and fall frequently appears near the crest of the curve, and sometimes in the "valley." The foregoing is the general character of the curve, but it is by no means regular, the tides on this part of the coast being governed largely by the wind. The tide lags irregularly behind the moon's phase, and much more than on the open seacoast, but the speaker has no data at hand on this subject. Mr. Pitts.

It is true that this work has stood for a comparatively short time only, as yet, but the speaker thinks that the results obtained since work on the jetty ceased, leaving it in an incomplete condition, are strong arguments in favor of the correctness of the theory on which it was designed.

Lewis M. Haupt, M. Am. Soc. C. E. (by letter).—The writer feels under obligations to his colleagues for the interest and trouble they have taken in discussing this paper, each from his own standpoint and experience, it is true, but these side lights furnish an opportunity to illuminate the general problem. Mr. Haupt.

General Review.—Some confusion has arisen from an attempt to measure results obtained elsewhere under totally different conditions; thus, instances are cited of the success of twin jetties where there is a large fluvial discharge at high velocities; cases which are not comparable with that under consideration where there is no land drainage and a very small diurnal tide. Again, it is attempted to compare a single jetty projecting from the shore with the reaction breakwater which is purposely wholly detached therefrom; or, still further, linear work, having but a single function to perform, is cited as a precedent for curvilinear work designed to develop diverse agencies.

Objections are also made to the use of the words "reaction breakwater" as being redundant, Major Symons not appearing to recognize the fact that the structure does break the waves from the north and northeast, forming a channel of quiet water in its lee and also produces a scour by the continuous reaction of the ebb currents along the concave front of the structure. These anticipated results, however, are so patent that no further space will be taken to reiterate them.

Mr. Corthell devotes considerable space to the impropriety of leaving an opening for the free admission of the flood tide, and quotes from Captain Eads the well-recognized principles applicable to half-tide or submerged jetties to sustain these views. As there is no question concerning their soundness and as the writer is on record in the papers referred to in his original paper, no time will be consumed in responding to this point which is also well covered by Mr. Marindin.

Mr. Haupt. Mr. Corthell recognizes that the theory is "exceptional" and "peculiar," the work incomplete, and that, if correct, the cost will be only about one-third of that under present plans. Certainly, then, it is worthy of demonstration, which is all the writer has been seeking for years, and the testimony of Nature's agencies are now produced, as far as they have gone, to establish the correctness of their application at this difficult locality. This testimony does not seem to be refuted by any of the arguments submitted. The writer does not claim universal application of his plans to all conditions as seems to be inferred from a few of the discussions, since each case is a special problem to be solved on its merits. Neither does he claim that a reaction is developed by a "convex bank." The function of the convex portion of the work is wholly distinct from the concave portion, but none the less important (as heretofore stated).

The unqualified statement that "it is not velocity * * *" that produces scour, is undoubtedly absurd, but it was not intended to be so used, as the context shows. It is a well-known physical fact that the mean velocity of a given stream, as generated by its surface slope, does produce remarkably different results under apparently similar conditions. The cases cited should suffice to illustrate the meaning, which was not to disparage either velocity or volume, without which no work could be done, but to direct attention to the tool, so to speak, whereby they may be applied to produce cutting. There is ample energy stored in the effluent at every bar, but, unless it is developed by some form of resisting medium and made to operate on the bottom, it glides to sea inoperative. The mean velocity is broken up and distributed into complex bottom movements by the resisting breakwater.

A serious defect, so oft repeated in harbor works, in the writer's opinion, is the manner in which it is attempted to apply straight jetties to develop a channel; with the result that a deep hole is scoured out at the point of incidence of a current and the energy of the stream is dispersed by being reflected at an angle instead of being continuously supported across the bar. This is observable in many instances where twin jetties have been placed. It is the purpose of the curved breakwater to maintain a continuous reaction; which has been done, as shown by the results already secured. The currents are moulding their own channel, with a thalweg about 500 ft. from the axis of the work and parallel thereto.

Dublin harbor, which is regarded as one of the most successful instances of convergent jetty scour, was visited by the writer in 1889, and described in the paper on "Jetties for Harbor Improvements." Here the benefit was alleged to be due largely to the 600-ft. gap in the North wall, which admitted more water at flood than it emitted at ebb, thus changing the equilibrium between the heads of the jetties slightly

in favor of the ebb. The dredging of the area within the jetties is not germane to the subject. From the relative positions of the jetties, shoaling is to be expected in this harbor. Mr. Haupt.

Neither does the writer regard it as important to reopen the discussion on the resultant direction of the littoral drift at Aransas, which was brought out by Mr. Sweitzer's paper on the "Origin of the Gulf Stream,"* to which attention is directed. Suffice it to say that the accumulation of drift on every jetty built in the Gulf furnishes suggestive evidence as to the correctness of the location of the breakwater.

The writer does not wish to disparage any carefully observed data, but in these problems the elements are so varied in different parts of the same inlet and are changing so constantly (every hour almost) that even a very large number of simultaneous observations, extended over long periods, will not furnish as authentic a record as the chronographic resultant engraven on the plastic material moulded by the forces themselves. Careful comparative surveys are indispensable for this purpose, as well as measurements of the forces which have effected the results. A study of such charts, at various dates, is of more value than a few instrumental observations of velocities of currents, winds or tides, all of which, however, have a relative value. The writer's conclusions, as to the resultant direction of the littoral drift, were more fully set forth in previous publications. It is largely a resultant of the angle of wave approach, configuration of shore line, tides, wind, etc., all of which co-operate to produce the effect. At Aransas the consensus of opinion, as well as the efficiency of the work done, point to the fact that the littoral drift is southwesterly. The small spur, 600 ft. long built in 1869, was a strong pointer.

Twin Jetties at River Mouths.—The writer makes no issue as to the use of parallel jetties at the mouths of rivers, as his paper states. Twin jetties are but the logical outcome of Nature's laws in attempting to conduct an effluent stream over or through an obstructing medium. They constitute an aqueduct. To be most successful, they should confine the land discharge and exclude the tidal waters. The greater the preponderance of outflow, the better the result. But, in sedimentary streams, jetties serve merely to conduct the flow across the site of the original bar. When it reaches the sea, and the velocity is checked, the sediment is again dropped and the process of bar building and jetty extension must continue, unless there is a sufficiently strong littoral current to conduct the silt away from the outlet. If not, the depth may be maintained by dredging. Hence, as stated, every problem has its local features. A single curved jetty at a river mouth, however, may suffice to sustain the currents while crossing the bar, and prevent deposit, so long as they are reacted upon, causing their load to be thrust aside beyond the influence of the reaction and thus

* *Transactions*, Am. Soc. C. E., Vol. xl, 1898, p. 86.

Mr. Haupt. building up a mud or sand jetty at a suitable distance for navigation and automatically adjusted to the regimen of the stream, thus saving the price of one of the twin jetties.

This action is well illustrated in the results at Aransas, where the sand bar has been thrust aside and moulded without advancing seaward, making a counterpart to the breakwater, and aiding in confining the currents to their natural path. Other instances will be cited later.

Brazos Entrance.—As attention is called to the depths stated at the mouth of the Brazos as "erroneous" the writer begs to add that he hoped to find decidedly increased depths from the excessive fluvial discharge of last July's floods, and hence wrote to Texas to learn the results. A perfectly reliable authority replied, as stated in the paper, and on this condition a comparison was predicated. That it is substantially correct is confirmed by the letter of Mr. Dorchester, cited by Mr. Corthell, stating that it had shoaled to "15 ft." (It must have been at this date that the inquiry was reported.) "Subsequently deepened to 18 ft." The letter adds: "Bar entirely outside of jetties," etc. This, then, is still an incomplete work, and no final conclusions should be drawn. The writer has always had confidence in the twin jetties, as designed and partially built for this, the most promising river in Texas, for the application of this method. It is somewhat similar to Tampico, where Mr. Corthell's work has proven entirely successful. No "attack" is made on old methods, where properly applied, but it should be remembered that science is progressive, and in suggested improvements defects of existing methods may, with propriety, be pointed out, for the purpose of emphasizing and illustrating the particular applications of the new. This does not mean an unqualified condemnation of the old, and is in strict conformity with the law of evolution.

As early as November 19th, 1888, Mr. Corthell very kindly received the writer's publications and plans, returning them with the remark:

"They are all very interesting and you have certainly done an immense amount of work in collecting data relating to harbor phenomena. I cannot concur with you, however, in your improvement of tidal harbors, and cannot bring myself to believe that detached works such as you propose will obtain the depths required. * * * I have no occasion to design any works for exclusively tidal harbors."

This was before any results were obtained by any works of this character; but, for lack of space, further consideration of this valuable contribution must be deferred.

Mr. Marindin, physical hydrographer of the Coast Survey, recognizes the fact that streams, in carving out their channels in alluvium, ignore straight lines (which are those of the greatest slope between given points in a bend), and take to the curves where they scour to greater depths, as cited also by Mr. Ripley. This furnishes another answer to the point raised as to the velocity due to surface slope

being the main factor in cutting channels, and brings out the reaction feature which has been so little utilized hitherto in engineering applications. Mr. Haupt.

New York Bay.—In referring to the "Narrows" of New York Bay to illustrate the effect of "reaction *versus* velocity," as a cause for the observed effect, the writer quoted from memory impressions gleaned some 12 years before from the reading of a manuscript report by Professor Henry Mitchell, on the physics of this bay. This report was made in 1858–59, and revised in 1862, but was not published. The following extract therefrom is made by the courtesy of Professor Mitchell:

"The greatest depth of water was found at Station G (Fig. 6) in the Narrows, and here our lowest point of observation was 92 ft. from the surface. At this depth we found the flood current prevailing ten (10)

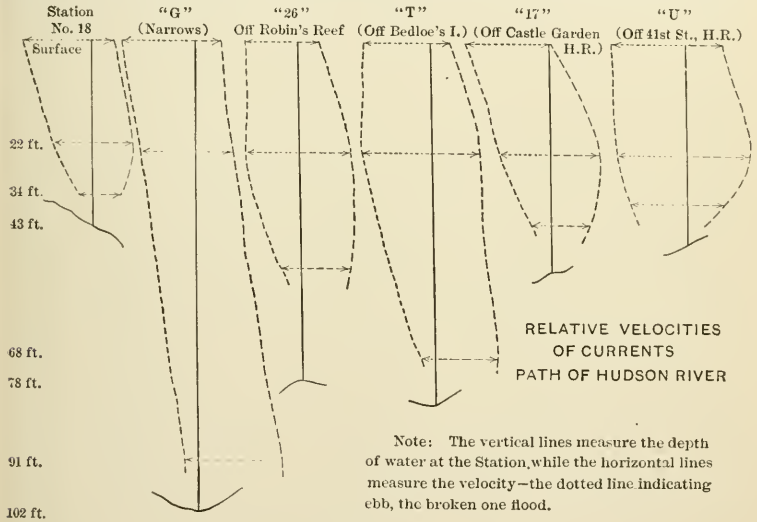

Station No. 18
"G" (Narrows)
"26" Off Robin's Reef
"T" (Off Bedloe's I.)
"17" (Off Castle Garden H.R.)
"U" (Off 41st St., H.R.)
Surface
22 ft.
34 ft.
43 ft.
68 ft.
78 ft.
91 ft.
102 ft.
RELATIVE VELOCITIES OF CURRENTS PATH OF HUDSON RIVER
Note: The vertical lines measure the depth of water at the Station, while the horizontal lines measure the velocity—the dotted line indicating ebb, the broken one flood.

FIG. 6.

hours out of the twelve (12) giving place to the ebb only at the most rapid fall of the tide. The flood stream, coursing along the bottom, reaches its maximum velocity near the time of high water; the highest rate observed being 1.82 miles per hour, a powerful scouring force! The greatest velocity of the ebb at the same point was observed to be 0.25 mile per hour, a force which according to the observations of Dubuat is incapable of overcoming the rolling friction of fine sand.

"The surface observations at this station indicate a very great preponderance of the *ebb current*, but it does not prevail so persistently as the flood in the lower stratum."

These resultants are more clearly illustrated by the diagram, Fig. 6, which accompanied this report, and which exhibit so forcibly the existence of this great vertical eddy swaying with the tide.

As the observations for velocity and duration were at a depth of 92 ft., while the bottom was 102 ft. below the surface, and the flood

Mr. Haupt. resultant increases with the depth, it is probably nearer the truth to place its duration at 11 hours on the bottom, as stated in the paper.

Is it safe to presume that a maximum velocity of 2.7 ft. per second, acting alone, will scour material from the bottom of a hole 102 ft. deep and will roll it up a slope to the level of the bed of the harbor, probably 60 or 70 ft. higher, when similar velocities will not disturb fine sand under 6 ft. of water on a down grade? Certainly, there must be some agency other than mere velocity to explain this phenomenon. The cautious engineer must take it into consideration, and may utilize it in designing his works.

Mr. Marindin is correct in his predictions as to possible extensions of the breakwater in case of extensive deposits beyond the end of the works, but as this is a drift and wave bar, with a practically clear effluent (excepting perhaps during Northers), and as the reactionary effect of the currents is to displace material in a lateral rather than a longitudinal direction, rolling it to one side before reaching the outer end of the breakwater, and as this end curves so as to create confluent streams at certain stages of the tide to maintain the movement of whatever sand may be carried out, there has been no appreciable deposit at the outer end of the work, nor any advance seaward of the bar. The results speak for themselves. See discussion by Mr. Pitts.

In a sedimentary stream, a different curve and location should be adopted, to admit of greater possible extensions. That this is an important factor will be seen from the rapid movement of the Galveston bar, resulting from the failure to arrest the littoral drift by the construction of the north jetty first; or still better, by the concave breakwater recommended by Mr. Ripley in 1883. The several efforts to secure deeper water at Aransas by a south jetty first also failed, because of the bar advance resulting in less depths. The effect is the same as that of placing a snow fence on the wrong side of a railroad cut.

The following is a statement showing the advances of the bar at Galveston, Texas, as indicated by the movement of the outer 18-ft. contour, and is taken from the blue prints of official surveys made on the dates indicated; measured from a "base" line joining station 220 + 18 on the North Jetty with the South Jetty beacon:

			From "base" to 18-ft. contour.
October	4th,	1894	650 ft.
"	23d,	"	750 "
December	"	"	1 150 "
March	3d,	1895	1 800 "
April	"	"	1 800 "
"	29th,	"	2 300 "
June	3d,	"	2 200 "
"	30th,	"	2 350 "
July	31st,	"	2 550 "
September	2d,	"	3 300 "
"	27th,	"	3 350 "

Depth in natural channel 16 ft. crossing south jetty. Over 60 000 ft. of jetty work in place in 1896. The increased depths in the channel were secured and maintained by dredging. The advance of the bar seaward during the year from October 4th, 1894, to September 27th, 1895, was, therefore, 2 700 ft. on a little more than half a mile, which was unprecedented. The bar, which was originally about 3 miles out, is now over 5, and still advancing. Mr. Haupt.

The writer coincides entirely with Mr. Marindin's closing remarks as to the use of parallel jetties at the Brazos, while the case of the South or South West Pass is too large a problem to be embodied in this discussion, and must consequently be deferred.

Mr. Wisner distinguishes clearly the necessity of adapting the remedy to the conditions as to tidal or fluvial embouchement; also as to the need of admitting the full tidal prism at flood, and confining it during ebb to a local path, thus changing materially the conditions of equilibrium over that part of the bar intended for navigation, which cannot be done at tidal inlets by two tight jetties. The attempt to accomplish it has resulted in the waste of millions of dollars and constant charges for maintenance by sluicing or dredging.

While twin jetties have been successful at the mouths of rivers having no estuaries, the writer does not think, as already intimated, that a single well-built and properly placed reaction jetty would not produce quite as good results where there is a resultant littoral current to remove the débris and prevent the advance of the bar. In fact, he is familiar with several very successful precedents. To cite only one, a single curved jetty of 3 000 meters radius has deepened its bar four-fold, where "for centuries before the struggle against the bar consisted in prolonging simultaneously two jetties from either bank of the river, leaving between them a distance of about 500 ft. But the sands outran the heads of the jetties, the flats were extended and the bar reformed. * * * Now (1889),* the bar has actually vanished," and the tonnage has increased enormously. From this instance it would appear that a single concave curved breakwater at the mouth of a river may produce the relief desired at much less cost than by two jetties, and hence becomes the better solution. This is also confirmed by the experience at the mouth of the Columbia River in Oregon, which was begun by Major Powell, Corps of Engineers, U. S. A., in 1885, at an estimated cost of $3 710 000 for a half-tide jetty, to be supplemented by raising it to high water, "or by building a short work on Peacock Spit for an increase of contraction."† The second jetty was not built, as it was found necessary to raise the first to high water to intercept the sand successfully, and this, with the drainage of the Columbia River, served to produce sufficient depth.

Mémoire, Société des Ingénieurs Civils, etc.

†In closing the discussion on Grays Harbor in 1896 it was stated that the writer was in error on the proposition to use two jetties at the mouth of the Columbia, and there was no opportunity to state the authority which is, therefore, quoted as above.

Mr. Haupt. Experience, thus far, at Aransas does not indicate that the short spur on the west side to the wreck of the *Mary* will ever be required, since the currents, even in the present condition of the work, are abundantly able to protect the channel from the eddy action referred to, and the fact that the breakwater is detached from shore modifies this action greatly. The currents have built their own protecting barrier of sand without cost to anyone or injury to navigation.

The jetties at Galveston, Cumberland Sound and other points are placed too far apart to co-act in producing scour. This results from the desire to admit as much of the flood tide as possible and therefore lessens greatly the concentration at ebb, and, since both flood and ebb movements follow the same path through the same area of cross-section, there is no beneficial resultant, and dredging becomes a necessity, while the bar moves seaward at a more rapid rate than before. This is not similar to the flood and ebb in a river where there is a resultant.

A few words as to Cumberland Sound, by way of illustration. The plan of 1879, as revised in 1891, consisted of two low jetties of stone, having their outer ends parallel and about 3 900 ft. apart. Work was commenced in 1881. The expenditure to July 1st, 1890, amounted to $479 550.28. The results were "considerable shoaling." The channel divided, the southerly one having "13 ft. at mean low water the same as reported last year."*

In reporting on this site, in 1879, General Gillmore said "the draught of water that can be carried over the Fernandina Bar (Cumberland Sound) varies from 11 to 12½ ft. at mean low tide, with exceptional depths 1 ft. greater, and a mean tide of about 6 ft." Estimated cost $2 071 023 to "give a low-water depth in the new channel of 20 to 21 ft."

The present depths between jetties, as shown by the chart of 1898† is less than 6 ft., and there has been appropriated $1 787 500. Hence it follows that for each of the, say, 6 ft. of depth lost between the jetties the cost has been about $298 000, while it was estimated originally that each foot of depth gained should have cost (estimating on 8 ft.) $260 000, making a difference of $558 000 per foot between the predictions and realities. The actual increase obtained by dredging south of the jetties, as reported, was 2 ft., making the cost per foot $893 750 for a temporary result secured after about nineteen years of effort.‡

On June 30th, 1897, the crossing on the bar half a mile south of the jetties gave "depths of only about 13 ft." Captain Carter then reported that the project has "proved unsuccessful for the sole reason that funds have not been supplied regularly and in adequate

* Report. Chief of Engineers. U. S. A., 1891, p. 1560, *et seq.*

† Report, Chief of Engineers. U. S. A., 1898, p. 1326.

‡ The cost of the dredging is not stated, but as it is not permanent and should be capitalized, say, at 3%, it will probably not differ greatly from $25 000, the interest on $833 333.

amounts," whereas, in the writer's opinion, the location and design of the jetties was largely responsible. Mr. Haupt.

In submiting his revised plan, dated October 8th, 1890, Captain Carter recommended a concave wall on the north side of the channel, thus intending to dispense with the southerly jetty. This, he says, would involve "the removal of the central portion of the South Jetty, which is improperly located." For this plan, including the removal of a portion of the South Jetty, the estimate was $1 108 000, while for the original plan the estimate was nearly double; yet, after revision, "the Board, after consideration of all the conditions, does not advise any radical change in the general direction of the jetties as originally proposed. It does not think that a single jetty on the north side of the channel, curving gently to the south, would secure the deep water needed, but is of opinion that two jetties will be necessary," etc.

In view of this opinion it may be instructive to add that the last report available (surveys of June, 1898) shows only 5.9 ft. between the jetties, instead of 13 ft., while the present channel crosses the south one, the outer end of which serves to protect a crossing which has been opened on the bar by dredging under its lee; so that the two jetties are absolutely useless for concentration and scour, serving merely as sand arresters, and are serious obstructions to the currents which they cross.

The project was revised again in 1895, and estimated to cost $2 350 000; and in 1897 it was recommended that dredging be commenced in accordance with the adopted project, $50 000 being appropriated for the purpose.* Work on the jetties, by the Atlantic Contracting Company, was suspended on October 9th, 1897. The dredge boat could not work between the jetties because of the shallow depth, so it worked south of the South Jetty. A clam-shell dredge was also put to work to open a channel through the South Jetty to a depth of about 15 ft. The next season a channel of 17 ft. depth was opened by dredging the bar up to the jetty, but it shoaled up during the following months. The results secured, therefore, are wholly due to dredging, and are temporary in character.

It remains to connect the recommendations of Captain Carter, of a single curved wall or breakwater in October, 1890, and its rejection by the Board, with the plans of the writer, by stating that these plans were mailed to Captain Carter, then in charge of Cumberland Sound, at his request, on May 5th, 1888, followed by the discussion of the adverse report of the Board on the writer's plans,† March 28th, 1889, yet, after a careful study of the local conditions, in the light of the facts therein contained, he (Carter), without the knowledge or advice of the writer, concluded to recommend their adaptation to his problem, but

* Report, Chief of Engineers, U. S. A., 1898, page 1324.

† "Discussion of the Dynamic Action of the Ocean in Building Bars, being a reply to the Report of the Board of Engineers, U. S. A.," *Proceedings*, Am. Phil. Soc., March, 1899.

Mr. Haupt. which the Board declined to endorse, with the results as above. It is probably this action which Mr. Wisner characterizes as "the deadly wisdom of the Engineer Board * * * which proved fatal to the project."

The opening paragraph of Major Symons' contribution creates the impression that he is unable to distinguish, as was also the Board of Engineers of 1889, between a reaction breakwater, placed on the crest of the bar and having a specific form and position, and an ordinary jetty having its root on shore and projecting as a groin into the sea; but the distinction is radical and vital, and it would seem that the mere assertion of the lack of originality, as an opinion, should have but little weight when not supported by reference to the alleged precedents, which neither he nor the Board have seen fit to quote. Neither has the writer, after diligent search through an extensive field of harbor literature, been able to discover any anticipation, even approximate, save that of Major Ripley as proposed, but never built, at Galveston.

In view of the claims made in this part of the discussion as to the single-jetty work done on the Pacific Coast by Major Symons, the writer feels constrained to state that it is believed all of his work was planned subsequent to the publication of the plans and the discussions provoked thereby.

The instance of Grays Harbor cited was not discussed until about nine years after the writer's earlier papers appeared, and if "it is in every essential respect such a reaction breakwater as described," as stated by Major Symons, then it does not furnish a precedent, but rather a remarkable confirmation of the soundness of the writer's plans, and becomes as well an infringement upon his property rights to his idea.

As the writer stated his views in the discussion of 1896, it will suffice to refer to the *Transactions** of this Society in reply as to the expectations; but the subsequent work done there will serve to show how little actual resemblance there is to the reaction breakwater, for the jetty is built on the south side, is connected with the shore, is straight for a considerable distance whence it curves away from the channel (or is convex, not concave), is intended to be $3\frac{1}{2}$ miles long, is estimated to cost $1 000 000 and is expected to produce 24-ft. depths. In other words, it is not solely a tidal harbor; no provision is made for changing the equilibrium of ebb and flood or for the development and utilization of the reaction of the ebb currents, while only a partial barrier is opposed to littoral drift by this structure, which is so different in plan and location from that under consideration.

The ruling depth shown in the chart of 1896 was 13 ft., and that in the report of 1898 remains the same, while the proposed jetty crosses

* *Transactions*, Am. Soc. C. E., Vol. xxxvi, 1896, p. 109.

the channel. It would therefore seem to be a repetition of the un- Mr. Haupt.
fortunate Government experience at Aransas Pass in 1885, excepting that the outer end of the jetty is convex to the channel instead of concave, which is so much the worse for the channel. At all events, since the jetty is not yet completed and no results are available, it does not seem to bear at all on the question of a precedent as to tidal scour resulting therefrom. This is confirmed by the admission that, after all, double jetties may be required there, and they are estimated to cost $2 500 000.*

The next reference given, as anticipating the writer's plans, is the single jetty at Coos Bay, located and built under the writer's direction and supervision, 1890 to 1895, which is a "reaction breakwater," etc. Here, again, there is a lapsus in the chronology, since the plans of the writer were published in 1887–88. How far, therefore, Coos Bay may be an anticipation or even an embodiment of those principles may be seen by a brief reference to its history.

The annual report of Captain Powell, 1886, says:

"The plan is to build a half-tide jetty or deflecting dike about 2 400 ft. long, * * * inside of the entrance, on a slightly curved line toward Coos Head. * * *"

The amount expended is $128 259.25. The original estimate of cost is $600 000.

This is, therefore, only a wing dam built on the bank of the stream inside of the bar. It was soon "submerged" and injured and "jetty operations were last closed in December, 1884."

At this point there is a rocky headland on the south and a sand spit on the north. It is said that "about once in 5 years the outflow sought a shorter line by a breach through the north spit," etc. Depth at low water, 14 ft. The report of 1898, made by Major Fisk, Corps of Engineers, U. S. A., says:†

"The present approved project‡ provides for constructing two, high-tide, rubble-stone jetties to obtain and maintain at the entrance to the bay a low-water bar depth of at least 20 ft. The North Jetty to be 9 600 ft. long and the South Jetty 4 200 feet long."

It also provides for sand fences on the north spit. Estimated cost, $2 466 412.20, independent of the $213 750 expended on the original project. "All operations have been confined to the North Jetty. * * *" The channel was well south of the jetty, but it has since moved gradually back to its normal position nearly parallel to the line of the jetty.

The map accompanying the report of 1891 § shows the plan to consist of two jetties, the north one extending from the sand spit southwardly, to canalize the effluent, then curving in a long sweep,

* Discussion by Mr. Allardt, *Transactions*, Am. Soc. C. E., Vol. xxxvi, 1896.

† Report, Chief of Engineers, U. S. A., for 1898, p. 2963.

‡ See Report, Board of Engineers, Officers, 1890, pp. 2936–65.

§ Part v, facing page 3162.

Mr. Haupt. convex to the channel and extending seaward in a straight, westerly line parallel to the companion jetty as proposed from Coos Head and 1 500 ft. distant therefrom. This chart bears the signature of Captain Symons, and is dated 1891. It contains the following legend:

"To construct two brush and stone jetties converging upon and crossing the crest of the bar at a distance apart of 1 500 ft. * * *"

Mean tide, 5 ft. 6 ins. Soundings in feet show a minimum depth of 19 ft., so that whatever results were secured at Coos Bay at that date were due solely to the renewal and extension of the Powell inner jetty of 1879, for which the twin jetties were substituted by the Board of 1889, and which were not in existence in 1891. Naturally, under this project, the proper order of procedure would be to build the North Jetty first, as was done, thus controlling the movements of the spit and protecting the channel; and this work, with the rocky headland, should suffice, if well placed, to maintain the requisite depth, with the assistance of the ample tide and rapid land drainage, so characteristic of the Pacific Coast. By this date, probably over a mile of this jetty has been completed, and to June 30th, 1896, the sum of $738 750 was appropriated, but the gain in depth resulting therefrom is not stated in this discussion. The Report of 1898 states, however, "During the past year the channel over the bar had a depth ranging from 18 to 22 ft. at mean low water," while the Report of 1892 also states that at various times there were depths of 18 ft., and, when the channel was 1 000 ft. further south, there were 27 ft. at low water across the bar. As there was 19 ft. of water shown on the chart of 1891 before the North Jetty was started, it is difficult, therefore, to see wherein it has materially aided in deepening the channel by natural scour, even with the large volume of tidal and fluvial waters available.

In short, the North Jetty as built at Coos Bay is in no sense a reaction breakwater, such as is partially completed at Aransas Pass, and the author fails to understand how a structure which did not exist in 1891 when the depths shown prior thereto varied from 19 to 27 ft. can be said to have developed a deep bar channel when the reported depths at date, 1898, were 18 to 22 ft.

Neither is the jetty at the mouth of the Columbia a "reaction breakwater," according to the writer's views and purposes. It is merely a sand barrier serving to arrest the littoral drift and thus reducing the work to be done by the ebb stream; but, as its outer end curves away from the channel, its reactionary effects are lost and its efficiency reduced. Moreover, the physical conditions as to its being exclusively tidal are wholly dissimilar.

The allusion to San Diego appears to be equally unfortunate as an attempt to establish a precedent. In 1891, it was reported that "The approved project contemplates the construction of a jetty about 7 500 ft. long on Zuniuga Shoals, so as to give a depth of 26 ft. at mean low

water over the bar, the present depth at the same stage being 21 feet."* Mr. Haupt.

There was already another but crooked channel, having 36-ft. depths, and it was proposed to create a new one in a different place by building a breakwater from Coronado Island to cut off the drift and to deepen to "24 ft." by dredging.

Work on the jetty was commenced in September, 1893, and the latest report at hand, 1898, states:

"No work was done during the fiscal year ending June, 1898. There is now an available depth of 21 ft. at mean low tide. * * * Total cost to June 30th, 1898, $289 741.35."

Hence there is no improvement in depth and no similarity of conditions to those at Aransas, while the entire project was executed subsequent to 1887.

On this basis this analysis might be extended indefinitely, but the writer believes that his statement, as made, remains unshaken; that the incomplete work at Aransas marks "a distinct advance in the resources of maritime engineers"; and he appreciates the force of the concession made by Major Symons in his opening paragraph that "some of the most successful works of the Corps have been accomplished by adopting in the main the principles enunciated by Professor Haupt," even though the references cited do not appear to show very great improvements, if any, nor an adaptation to any considerable extent of the principles. But the intentions are unquestionably good, even though the principles may not have been correctly applied.

In reference to the alleged precedents on the North Pacific Coast it should be noted, moreover, that Major Symons states the movement of the channel at Coos Bay was to the "Northward," and hence the jetty was placed on the north or far side thereof, while at Grays Harbor the "channel had an unswerving tendency to move from north to south, etc." "The jetty was located to interpose itself against this southerly movement with the hope and belief that the pressure to the south would continue." This jetty was, therefore, placed on the far or south side of the channel. Thus, in both instances, they are directly opposed to the location as adopted at Aransas, where the jetty was purposely placed between and not beyond the resultant littoral and the channel, with such decidedly beneficial results, while, in all other cases of Government work on the Gulf, the first jetty was constructed on the far side of the natural channels, with disastrous results and greatly increased cost.† This erroneous location, on the far side, increases the work of the ebb instead of diminishing it, by permitting the littoral forces to carry material into the channel instead of arresting it on the near side, which is a funda-

* Report, Chief of Engineers, U. S. A., for 1891, p. 2961.

† *Vide* discussions on Sabine Pass, Galveston, etc., in *Journal*, Franklin Institute, and in *Proceedings*, Am. Soc. C. E.

Mr. Haupt. mental condition of the design and location, which does not appear to have been fully comprehended or applied by the engineers of those works. Yet the writer has laid it down as the first desideratum, viz., to keep the littoral drift from entering the channel. A jetty on the far side does not meet this condition, and is therefore no anticipation of either the principle or its application.

While claiming similarity, Major Symons goes on to show radical differences, and the reasons therefore. He calls attention to straight jetties built ostensibly to cause erosion and yet having "groins on the channel side to protect them from undue erosion," thus destroying in large part their efficiency. That the work at Aransas has withstood storms for over three years without material subsidence is a sufficient answer to the query as to how it was built. That was one of the purposes of its plan, and with the submerged Government jetty reflecting the currents against its face it may well excite surprise that it has not been undermined long ago. The gaps are undoubtedly a serious menace to the work. They were left by the Harbor Company and were beyond the control of the engineers, yet notwithstanding the losses of water and entrance of sand through them, the deepening of the channel has been continuous. To build the sill *G E*, as suggested by the Board of Engineers, and endorsed by Major Symons, would, to a large extent, destroy the preponderance of the ebb movements which the writer desired to create; cause the groin to fill with sand, the shore line to extend seaward, to deliver its drift in front of the channel, and re-create the bar in front of the breakwater. All of which are to be avoided as injurious.

The opening at the shore end was commended by H. L. Marindin, M. Am. Soc. C. E., Assistant on the Coast Survey, as early as December 26th, 1889, when he reported to the Superintendent, as follows:

"One of the good features in Professor Haupt's theory is his objection to the closure of beach or flood channels. This is a strong point in his system and it is in contradistinction to the plans generally submitted by engineers."

This strong point the Board's plan would destroy. Mr. Marindin evidently has seen no reason to modify this opinion since. In none of the cases cited by Major Symons is this salient feature present, neither is the concavity for reaction; nor the peculiar combination of curves intended to reinforce and protect the work from wave action on the one side or too great erosion on the other.

The writer does not claim that the combination, as partly built at Aransas, is a panacea for all ills nor that all its features are at all times "essential," but for tidal inlets he does believe that the results, under peculiarly adverse circumstances, stand unrivalled in the history of maritime works, and that its intelligent application will effect far more rapid and permanent results at much less cost than twin jetties as now applied.

Mr. Haupt.

If the writer comprehends clearly the drift of the argument submitted by Major Ripley it is to show that the principles embodied in the peculiar structure built at Aransas Pass are not novel, but, as the writer has made no claim to having discovered new laws or applied new principles, he need make no other comment than to call attention to the new embodiment of these old principles in the breakwater which was located here with the assistance of Messrs. Ripley and Wisner.

In the ancient harbor of Swinemünde referred to, however, there will be found some material differences between the curved jetties in pairs at the mouths of sounds having large fluvial water compartments with several inlets and the case in point. The extract from General Gillmore's report of 1881 shows two jetties, both curved and rooted to the coast line, which it is asserted were "entirely successful," yet "requiring the frequent use of dredging machines." This, certainly, is a very different construction and result from that at Aransas, where there has been a marked increase from a single detached structure, without dredging, and which has been constantly deepening.

The progressive improvement at Aransas, due to scour alone, may be seen from the following surveys and reports:

	Depth on Bar. M. L. W.
Survey of December 10th, 1896	6.5 ft.
" " February 2d, 1897	8.0 "
Old jetty breached (500 ft. in length) between December, 1896, and May, 1897:	
Survey of June 8th, 1897	8.75 "
" " November 5th, 1897	9.25 "
Reconnaissance, February 5th, 1898	10.00 "
Pilots reported, June 15th, 1898	11.00 "
" " August 29th, 1898	12.00 "
" " (by telegram) January 4th, 1899	13.3 "
United States Coast Survey, February 11th, 1899	15.0 "

These are the depths entirely across the bar, with depths on both sides of the old Government obstruction of over 22 ft.

The writer would be pleased to go more fully into the details of the harbor works on the Baltic, and make public some very interesting old charts, covering several centuries, secured during his visit to these ports, but the length of this discussion is already too great. Suffice it to submit a few excerpts from the report of Assistant Davidson, in 1875, showing the local conditions at Swinemünde. From this it appears that the longer mole is on the side to "windward" as regards the drift and not to leeward; there is also the usual phenomena attending jetties in pairs, but the channel is more permanent and more easily maintained because of its curvature, being concave to the channel.

Mr. Haupt. *Swinemünde on the Baltic.**—The Sound has three outlets. There is no tide in the Baltic save that created by winds. In 1828, the depth was 6 ft. There is a constant current out from Stettiner Sound, supplied by fresh water from the rivers Oder, Ihna and Ucker. The channel at 24 ft. depth is not quite 175 yds. (525 ft.) wide. Two ocean steamers can pass without difficulty.

"The artificial channel is composed of two nearly parallel moles of unequal length * * * curving well to the westward. The outer mole is 115° of a circle having only 2 000 yds. (6 000 ft.) radius, and for two miles the deep water scours directly under its wall."

The west pier has no parapet. The gales of 1874 and 1875 carried away 250 ft. of the east parapet bodily into the channel. The piers have been prolonged several times as the deposits have taken place. The beach has made 1 050 yds. seaward. What littoral drift there is seems to prevail from the eastward, and the deposit on the westward is partly from the Sound and partly from the littoral drift, but the increased depth is not wholly due to the piers, but in part to the constant use of a small dredger. The cost is quite moderate and the results the most favorable yet seen upon a coast subject to heavy gales.

In conclusion, therefore, the writer is unable to find, in the cases cited, a solitary instance of a tidal entrance where there is a detached, concave, single, reaction breakwater, which has effected so great scour, in so short a time, with no assistance from dredging, in the face of an almost fatal obstruction, and at so little cost. If such a precedent is known to exist he would be grateful to learn of its location.

In view of the results already attained by this reaction breakwater and of the efforts which have been made, first, to prevent its introduction; and, subsequently, to so modify it as to destroy its practicability, he believes that the work should be carried to completion upon the original plan as a contribution to science and a demonstration of its ultimate efficiency. It is self evident that if equally good results can be obtained by so inexpensive a work, without dredging, many millions of dollars may be saved, while the barriers which have so long obstructed our commerce may be removed much more rapidly.

* Notes from the Report of George Davidson, Assistant, U. S. Coast Survey, 1875, p. 300.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

Note.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

ELECTRICITY VERSUS STEAM FOR BRANCH RAILROAD LINES.

Discussion.*

By Charles H. Davis, M. Am. Soc. C. E.

Mr. Davis. Charles Henry Davis, M. Am. Soc. C. E. (by letter).—While the subject for discussion seems to be confined to the application of electric motive power to branch steam railroad lines, the members speaking on this subject, at the Annual Convention, June 30th, 1899, are evidently interested in the problem as a whole and have not confined themselves to "branch lines." Claiming the same privilege, the writer transmits the following as possibly throwing some light on the future development in the use of electric traction, not only on existing steam railroad lines, but for new high-speed interurban transportation systems.

Reversing the usual order, some of the writer's conclusions are given prior to the discussion, in order that each step may be followed more readily. The figures and tables may be found to have some errors (of which the author will be glad to be advised), and, in many cases, are only approximate; so that they must not necessarily be treated as accurate in their details, but they are safe as to their conclusions, and can be relied upon in this respect.

Conclusions.

(*1*) Steam railroads will, in the near future, handle their suburban and short-distance interurban passenger traffic and mail, express, bag-

* Continued from August, 1899, *Proceedings*.

Mr. Davis. gage and light local freight carried in said suburban and interurban passenger trains, by electric motive power; and this, irrespective of whether operating expenses are affected favorably or unfavorably.

(2) Steam railroads will not, in the near future, handle their freight traffic (other than mail, express, baggage and light local freight carried in suburban and interurban passenger trains), and long-distance passenger traffic by any other motive power than steam locomotives.

(3) Steam railroads may, under exceptional conditions of large volume and great density of passenger traffic over comparatively short distances, handle it by electric motive power, but such cases will be infrequent.

(4) New railway lines, connecting very large centers of population, where frequent service at much higher speeds than can be attained now by steam locomotives on existing lines are conditions of success, will be operated by electric motors.

There are three conditions under which suburban and short-distance interurban traffic will be handled profitably by steam railroads converting to electric traction:

1. (a) Where units can be light and frequent, and operated over comparatively short distances.

(b) Where gross receipts can be so increased by the change of system and mode of operation as to pay for the increased investment and possible increase in operating expenses.

(c) Where competition of parallel electric roads compels the change, to save what traffic there is, irrespective of how operating expenses are affected.

In the future development of steam railroad systems they will eventually be operated jointly with surface electric railways, either through actual mutual ownership, or by traffic contracts, leases, etc.

The above conclusions are obviously dependent upon what Mr. Prout properly defines as "traffic conditions," and not primarily engineering details or operating expenses.

Laws of Passenger Traffic.

The laws of passenger movement are not well defined, and many of them are illusive and hard to determine; the causes of loss or gain are often largely a matter of individual judgment, so that the following outline of them must not be taken as in any way exact.

People travel from one place to another from (1), necessity, and (2), pleasure or whim.

They are induced to travel more or less often according to:

1. *Total Cost from Point of Departure to Objective Point and Return to Place of Beginning.*—As the total cost is reduced, travel, due to both fundamental causes, is increased; presumably less rapidly than the

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EXPLANATION

Curve		Multiplier for Vertical Scale
1.	Total Miles of Track	1 000
2.	Annual Increase of Mileage	100
3.	Total Population of the United States	1 000 000
4.	Percentage of Increase of Population of United States	1
5.	Density of Population of the United States	1
6.	" " " " Massachusetts	2
7.	Total Urban Population of the United States	100 000
8.	Percentage of Urban to Total Population of the United States	½
9.	Total Population of New York, New Jersey and Pennsylvania	100 000
10.	" " " Massachusetts	100 000
11.	" " " New York City	10 000
12.	" " " Brooklyn	10 000
13.	" " " Philadelphia	10 000
14.	" " Served by N.Y.& P.R.T.R.R.	100 000

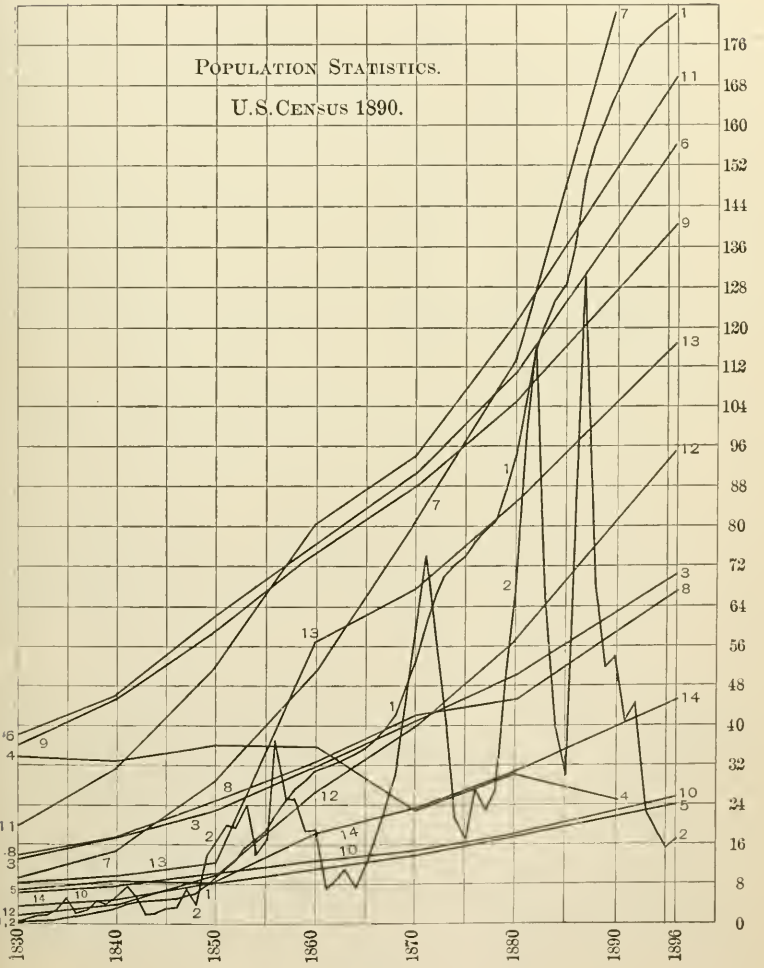

FIG. 1.

Mr. Davis. total cost falls. Note that the important factor to the passenger is total cost, and not cost per mile traveled.

2. Total Time Consumed in Making the Round Trip.—As the total time consumed is reduced, travel, due to both fundamental causes, is increased, presumably less rapidly than the total time falls. Velocity of transportation is not primarily effective in inducing travel, for it makes no difference to the passenger whether he be carried 80 miles in 30 minutes or only 12 miles.

3. Total Conveniences Afforded the Passenger.—These may be divided into:

(*a*) Proximity of departure and arrival points to possible passengers. As a "leave-at-your-door" service is approached, passenger traffic increases, but according to no known ratio to distance. Wellington laid down an approximate rule of loss of natural revenue for steam railroads of 10% per mile of removal from center of population as a minimum, 25% per mile as an ordinary maximum and a much larger percentage of loss, or even total loss, under certain conditions. Electric street railways have profited more by this kind of service, which they offer the public, than from any other reason; in furnishing it they give frequent and quick service, both of which are of the greatest importance in their effect on passenger traffic. Much less than a mile, however, will make or ruin the passenger traffic of a street railway.

(*b*) Frequency of the service. As the number of trips increases, so will the passengers, but less rapidly than the headway is shortened. A frequent service means less "total time" consumed.

(*c*) Character of terminals, stations, roadbed, equipment, and, in fact, all physical characteristics. That transportation system which offers, at the same rate and time, better physical conditions, which give comfort or even luxury to the passenger, will not only secure competitive traffic, but induce that which would not otherwise exist.

4. Total Population.—As the population served increases, the passenger trips per capita per annum increase, and somewhat faster than the inhabitants, unless modified by density and distribution.

5. Density.—As the density increases, it is probable that the rides per capita per annum also increase, but whether more or less rapidly is uncertain.*

6. Distribution.—A long, narrow town will give more rides per capita per annum than a square town having the same population.

* In an earlier publication the writer expressed the opinion that the increase was less rapid than the density, but wishes to leave this open for further investigation.

EXPLANATION.

Curve.		Multiplier for Vertical Scale.
1.	Total Miles of Track	10
2.	Annual Increase of Mileage	10
3.	Total Capital Stock, in Dollars	500 000
4.	" Bonded Debt " "	500 000
5.	" Capitalization " "	500 000
6.	" " per Mile of Track, in Dollars	500

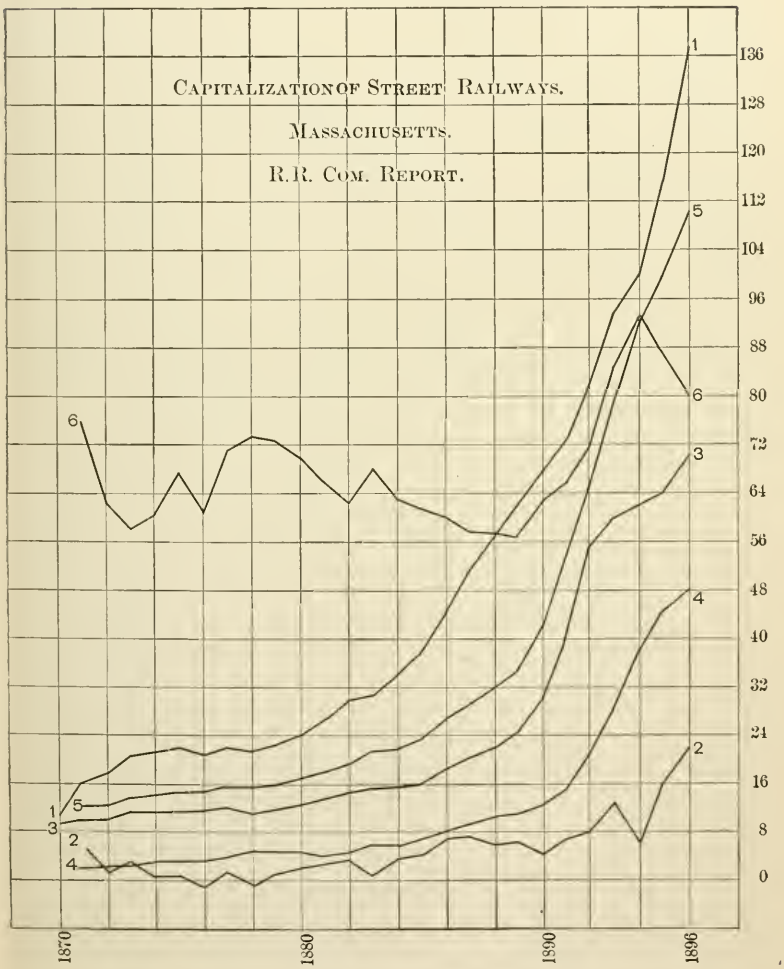

FIG. 2.

Mr. Davis.

7. *Character of the Industries and Population.*—The effect of various industries and the kind of population must have a decided effect on the passenger traffic, but probably according to no fixed laws, and certainly according to no known laws.

No "detailed defence" is offered for these laws other than what support they may obtain from the following discussion.

Any management of a steam railroad, in considering the question of adopting electric traction, will naturally turn first to data on operating expenses, for, if its use should lower the cost per train or car-mile enough to more than pay interest on the investment, then it would be of advantage, even though there were no other inducements, such as increased traffic. While comparisons might be made between the operating expenses of steam railroads, which are now well known, and such electrically operated roads as the Chicago Elevated Railways, or the third-rail lines of the New York, New Haven and Hartford Railroad, yet there are disadvantages in doing so which outweigh the apparent one of more similar conditions than exist between the ordinary street railway and the steam railroad. If it should appear that the operating expenses of electric street railways per car-mile are less than the cost per car-mile operated on steam railroads, one can be certain that electrical operation, applied to the latter, will result in reduced expenses. Whether or not this reduction will be enough to pay for the change is another question. For these reasons, a comparison between the street railways and steam railroads of the New England States has been taken as a basis in this discussion.

To exhibit the relative merits of the two systems of installations, the following divisions are arranged in tabular form and are treated in order. The prefixion of the word "electric" or "steam" indicates that the item belongs exclusively to one or the other system and has no counterpart in the other.

In comparing the steam and electric roads of New England, we are more likely to arrive at correct conclusions, as the population is more dense, the miles of road greater per square mile, there are many more examples to study, and the statistics are more accurate and reliable, and cover longer periods. The word "Railroad" is used to designate a "steam railroad," while "Railway" always indicates an "electric street railway," following Massachusetts precedent; the difference being that the former operates on its own right-of-way, while the latter occupies the public highway. This distinction is a desirable one to cultivate, and should be extended and used irrespective of the mode of operation; thus, a road operating by steam, electricity or any other motive power on its own right-of-way is a "railroad," and one operating on a public highway is a "railway."

EXPLANATION.

Curve.		Multiplier for Vertical Scale.
1.	Total Miles of Track	100
2.	Annual Increase of Mileage	10
3.	Total Capital Stock, in Dollars	10 000 000
4.	" Bonded Debt " "	1 000 000
5.	" Capitalization " "	10 000 000
6.	" " per Mile of Track, in Dollars	1 000

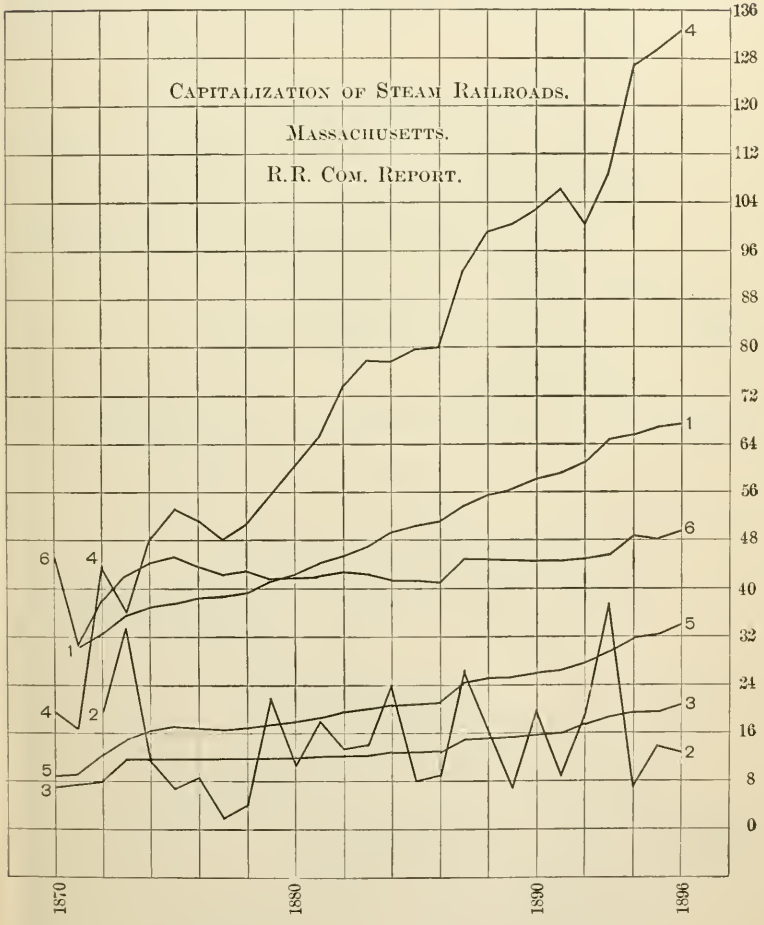

FIG. 3.

Mr. Davis.

TABLE No. 1.

1. FIRST COST.

(*a*) Right-of-Way and Real Estate.

(*b*) Construction.

1. Engineering and surveying.
2. Clearing.
3. Grading, ditching, etc.
4. Tunnels.
5. Masonry, culverts, bridges, trestles.
6. Fencing, cattle-guards, road-crossings.
7. Ballasting.
8. Ties.
9. Rails.
10. Track-laying, lining and surfacing.

(Electric) 11. Rail bonding.

(Electric) 12. Overhead-trolley line, or third rail.

13. Signals.
14. Telegraph.
15. Stations and all buildings (including building for central power stations of electric roads).
16. Terminals.

(*c*) Equipment.

(Steam.) 1. Locomotives and tenders.

(Electric) 2. Motor cars or electric locomotives.

3. Passenger cars.
4. Freight cars.

(Electric) 5. Electric feeder lines and ground-return circuit.

(Electric) 6. Central power stations (not including buildings which are considered under "Construction").

(*d*) General.

1. Discount on bonds.
2. Interest on bonds, to opening of road.
3. Taxes, to opening of road.
4. Office expenses, salaries, etc., to opening of road.
5. Contingent and miscellaneous, not itemized, to opening of road.

2. TOTAL EXPENSES.

(*a*) Maintenance and Renewal of Way and Works.

1. Repairs of earthwork to subgrade.
2. Repairs of track.
3. Repairs of fences, cattleguards, crossings, etc.

(Electric) 4. Repairs of ground-return circuit.

(Electric) 5. Repairs of overhead-trolley line, or third rail.

(Electric) 6. Repairs of electric feeder lines.

7. Renewals of rails.
8. Renewals of ties.
9. Renewals of ballast.

(Electric) 10. Renewals of poles, or insulation of third rail.

(Electric) 11. Renewals of trolley, or third rail.
(Electric) 12. Renewals of feeders.
13. Repairs of masonry, bridges, trestles.
14. Repairs of buildings.
15. Repairs of signals.

(*b*) Train Expenses.

(Steam) 1. Fuel for locomotives.
(Electric) 2. Fuel for power stations.
3. Water supply.
4. Oil and waste.
(Steam) 5. Repairs and maintenance of locomotives and tenders.
(Electric) 6. Repairs and maintenance of motor cars or electric locomotives.
7. Repairs and maintenance of passenger cars.
8. Repairs and maintenance of freight cars.
9. Use of foreign passenger and freight cars.
(Steam) 10. Locomotive services (wages).
(Electric) 11. Motor car or electric locomotive services (wages).
12. Passenger-train services (wages).
13. Freight-train services (wages).
(Electric) 14. Repairs and maintenance of power stations.
(Electric) 15. Power-station services (wages).

(*c*) Station, Terminal, Taxes and General Expenses.

1. Agents and station services (wages).
2. Station supplies.
3. Telegraph.
4. Taxes.
5. General officers and clerks.
6. Legal.
7. Insurance.
8. Stationery and printing.
9. Agencies and advertising.
10. Contingent and miscellaneous, not included above.

(*d*) Accidents, Loss and Damages.

1. To freight.
2. To passengers.
3. To property owned.
4. To property not owned.

(*e*) Interest Account.

(*f*) Fixed Charges.

1. Interest on bonds.
2. Rentals.

(*g*) Dividends.

3. GROSS RECEIPTS.

1. Passenger receipts.
2. Freight receipts.
3. Miscellaneous receipts.

Mr. Davis.

1. First Cost.

We shall consider a new line under steam-railroad conditions, first, and then a change of system on an existing railroad.

The first cost of (*a*) "Right-of-Way and Real Estate" cannot be affected by any particular motive power unless the use of electric traction permits the use of sharper curves and heavier grades, thus avoiding large cuts or fills. While a coupled locomotive undoubtedly increases "curve resistance," this is not sufficient to be of any great moment. If electric motors are applied to every car axle, then trains of the same weight, moved by electricity, can surmount heavier grades than with locomotives, or can pass over the same grades at higher speeds. Hill climbing is a matter of traction between driven wheels and rails, and the sustained horse-power of the motor or locomotive. With the power applied to every axle, there is no part of the load which remains unutilized for traction, but this cannot be done by the application of steam, and has, as yet, been only partially successful with electric motors. With the latter, their maximum power can be called upon for an indefinite time, because they do not give out at the top of a long, heavy grade. This advantage of electric traction is, however, only apparent; first, because long trains are not yet satisfactorily controlled where motors are put on every axle, and short trains of two to five cars have only half the axles supplied with power in present practice; and second, electricity being confined to suburban and interurban short hauls between large centers of population, the additional cost of avoiding heavy grades is comparatively small, and, furthermore, if they are not avoided the loss in time would result in a correspondingly larger loss of revenue.

Under (*b*) "Construction" we find that items 11 and 12 apply to electric traction only, while there are no items applying to steam-locomotive traction only. These items might have been placed under the heading "equipment," as part of the things necessary in electric traction to replace steam locomotives, and which could be added to motor cars or electric locomotives as equipment for an electric road. This phase of the question will be discussed under the heading "equipment." The engineering of an electric line might cost more than that of a steam road, but not necessarily; if the chief engineer was a competent railroad man with a thorough knowledge of electrical engineering, the cost would be the same; otherwise it would be increased by that amount paid an assistant engineer with special electrical knowledge, unless the line was costly enough to warrant several assistant engineers, one of whom could be versed in electrical matters.

The cost of surveying work, consisting of reconnaissance, exploration line ("shoo-fly"), preliminary, location, and their modifications or extensions, depending upon the importance of the line, its cost,

Mr. Davis.

EXPLANATION

Curve.		Multiplier for Vertical Scale.
1.	Gross Traffic Earnings, in Dollars.	100 000
2.	Total Interest on Bonded Debt, in Dollars	10 000
3.	" Dividends on Capital Stock, in Dollars	10 000
4.	Gross Traffic Earnings per Mile of Track, in Dollars	100
5.	Percentage of Expenses to Gross Earnings	1
6.	" " Interest paid on Bonded Debt	1/10
7.	" " Dividends " " Capital Stock	1/10
8.	Gross Earnings per Revenue Train-Mile (Car-Mile), in cents	1
9.	" Expenses " " " " " " "	1

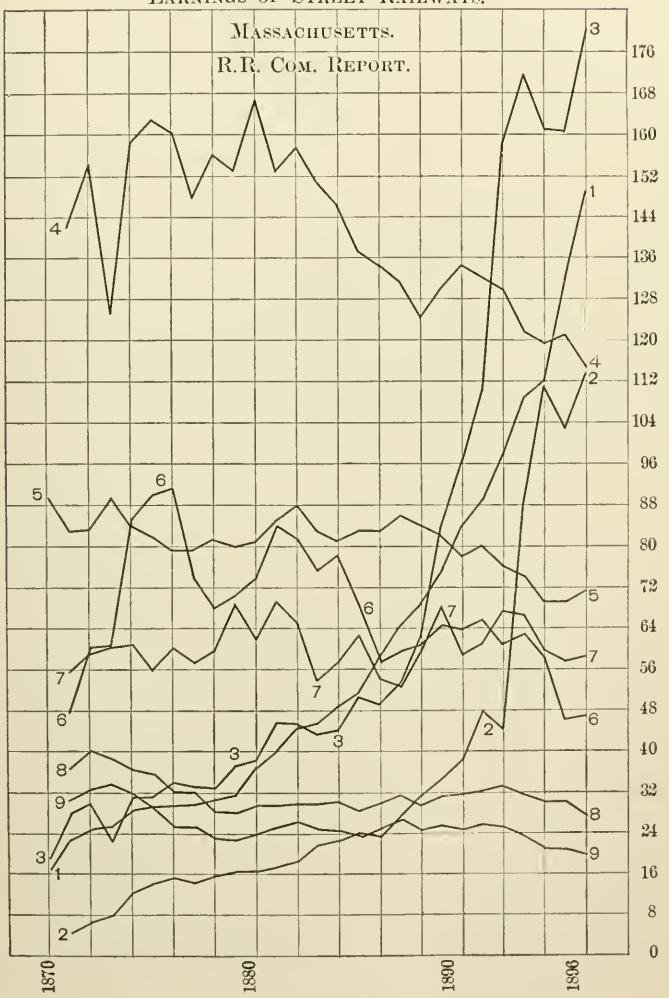

FIG. 4.

Mr. Davis. character of city and country, etc., would be the same in either of the cases under discussion for any specific example. The cost of clearing, grading, ditching, tunnels, masonry, culverts, fencing, cattle-guards, and those parts of the system pertaining to these general items, is not affected by the choice of system. It might be thought, at first, that the cost of masonry would be affected, if lighter bridges and trestles could be used with electric traction, but the weight and size of masonry abutments, piers, footings, etc., would hardly be affected by any slight changes in the moving load, for they are much more dependent upon the weight of the earth embankment which they support, and the character of the ground upon which they are built, together with their height and length. If they perform all their other functions, according to the best engineering practice, they will usually be found far more substantial than, theoretically, they need to be, in order to uphold the weight of the superstructure. Road crossings will be the same: (the additional cost of connecting a "broken" third-rail system being included under item 12.)

In the design of bridges and trestles, the principal elements which determine their cost are the span, dead weight, live uniform-load, concentrated live-load at head of trains (locomotive and tender), the speed of trains, and the "hammer-blow" of the reciprocating parts of a steam locomotive, especially when it coincides with the period of oscillation of a bridge (very objectionable on bridges which are too light). Electric motors, either geared to the axles or gearless (concentric with the axle), have a rotative motion, and therefore do not produce the "hammer-blow." Owing to the peculiar construction of the electric locomotive or motor car, a greater proportion of the total weight is on the driving wheels than is the case with steam locomotives; therefore, with the same tractive effect (weight on drivers), the use of electric locomotives or motor cars reduces the moving weight at the head of the train, and the weight of the tender is saved by the use of electricity. These three facts, theoretically, would enable bridges and trestles to be built lighter for roads using electric motors, providing the loads were not concentrated upon a smaller wheel base, were it not for the fact that the necessity of providing for constantly increasing weights practically offsets the theoretical saving which might be made. If motors were used on every car of the train, such structures might be designed so as to be lighter and less costly, but, for reasons stated, this is unlikely. In addition, if freight service should continue to be handled by locomotives, on the same tracks, or adjoining ones, lighter bridges would not be possible.

The cost of ballasting, ties, rails and track-laying, lining and surfacing will be the same in either case (drilling rails for bonding is covered by item 11), although a small saving in maintenance of these items is possible for electric traction (discussed under "total

EXPLANATION.

Curve.		Multiplier for Vertical Scale.
1.	Gross Traffic Earnings, in Dollars	1 000 000
2.	Total Interest on Bonded Debt, in Dollars	100 000
3.	" Dividends on Capital Stock, in Dollars	100 000
4.	Gross Traffic Earnings per Mile of Track, in Dollars	100
5.	Percentage of Expenses to Gross Earnings	1
6.	" " Interest Paid on Bonded Debt	1/10
7.	" " Dividends " " Capital Stock	1/10
8.	Gross Earnings per Revenue Train-Mile, in Cents	1
9.	" Expenses " " " " " "	1
10.	Percentage of Passenger Earnings to Gross Earnings	1/2
11.	" " Freight " " " "	1/2
12.	" " Interest and Dividends paid on Capital Stock and Bonded Debt	1/10

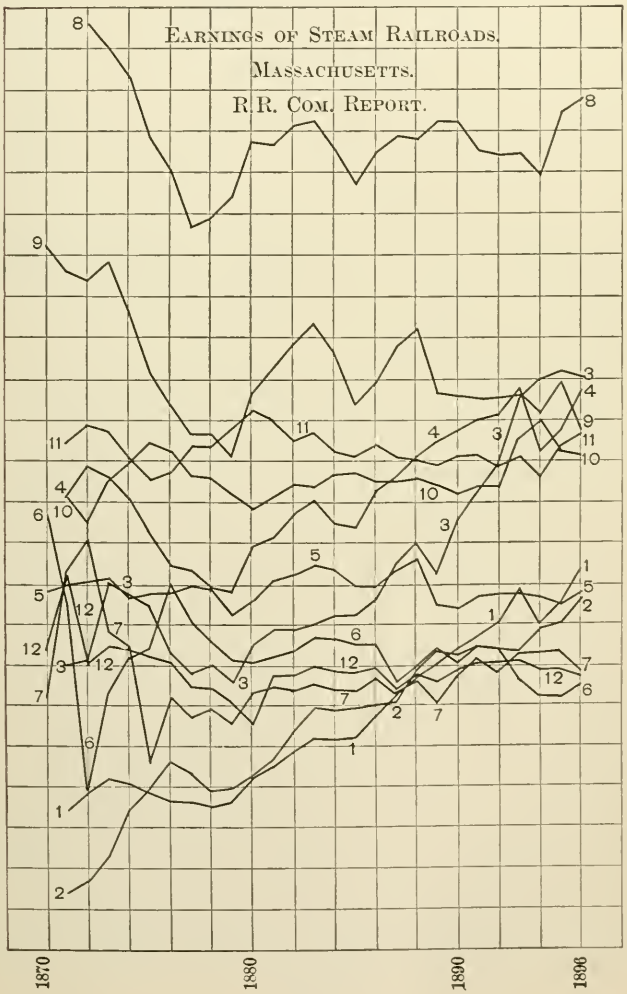

FIG. 5.

Mr. Davis. expense"), which indicates that the first cost could be reduced for the same cost of operating; yet, as this would be poor economy, the quality, weight and general character should be the same in either case. By the use of electricity it might, at first, appear that the signal and telegraph system would be reduced in first cost, but the only possible saving would be in the source of the current used, which forms so small a percentage of the whole that for our purposes any difference can be neglected.

Stations would cost the same under similar conditions, whether steam or electricity were used, although the electric road requires buildings for its centralized motive-power plant (that is, all but the transmission line and motors) which are not a burden on the steam road, yet these are no doubt balanced by round houses, coal stations, water stations and the larger machine shops required for steam locomotive repairs. Terminals will cost the same, whichever system is used.

So far, we have found that the first cost of construction is substantially the same, whether steam locomotives or electric motors are used, but, if the latter are adopted, some additional work is necessary, namely (11), rail bonding and (12), overhead-trolley line, or third rail. It will at once be seen that these items add considerably to the cost of construction of an electric road, as compared with that of a road operated by steam locomotives.

As an example, which will show, in a general way, the increased cost of electric-traction construction, let us assume, without any pretension to exactness, for each individual case will differ in the amount of increased cost, that the line is single track, one mile long; average speed of trains (including stops) 20 miles per hour; headway, one minute; length, six cars (one being a motor car); total weight of train, 200 tons when loaded to ultimate capacity; rails, 90 lbs. per yard; average nominal horse-power, 300 per train (will exert 450 H.-P., or even more, if required, for short periods); total maximum average nominal horse-power per mile of track (one way) at above headway, 900; average loss in transmission, 20%; assumed average nominal horse-power at station, 1 125 (allowing for loss and including surplus power), located at one end of the line. With these assumptions, we have the following approximate additional cost for construction, if electricity is used. The electric return circuit would consist of the two lines of rail properly bonded with copper wires, two 4/0 B. & S. G. wires to each rail joint, which would cost (copper, 15 cents per pound), complete in place, including terminals and drilling rails, 70 cents each bond, or a total of $497. To this must be added the cross-bonding of the lines of rails, which varies much in practice. For our case, we assume that this is unnecessary, if the bonds at the joints are properly made and maintained with the care displayed on other parts of the system. The overhead-trolley line would be of iron

Mr. Davis.

EXPLANATION.

Curve.		Multiplier for Vertical Scale.
1.	Passenger Train-Miles (Car-Miles.)	1 000 000
2.	Total Passengers Carried	2 000 000
4.	Gross Passenger Earnings. (Same as Curve 1, Fig. 4)	
6.	Receipts per Passenger Carried, in Cents.	1/10
7.	" " " Train-Mile. (Same as Curve 8, Fig. 4)	
8.	" " Mile of Track. (Same as Curve 4, Fig. 4)	
9.	Number of Passengers Carried per Mile of Track.	2 000
11.	Passenger Rides per Capita per Annum	1
12.	Receipts per Capita per Annum, in Dollars	1/10

PASSENGER TRAFFIC OF STREET RAILWAYS.

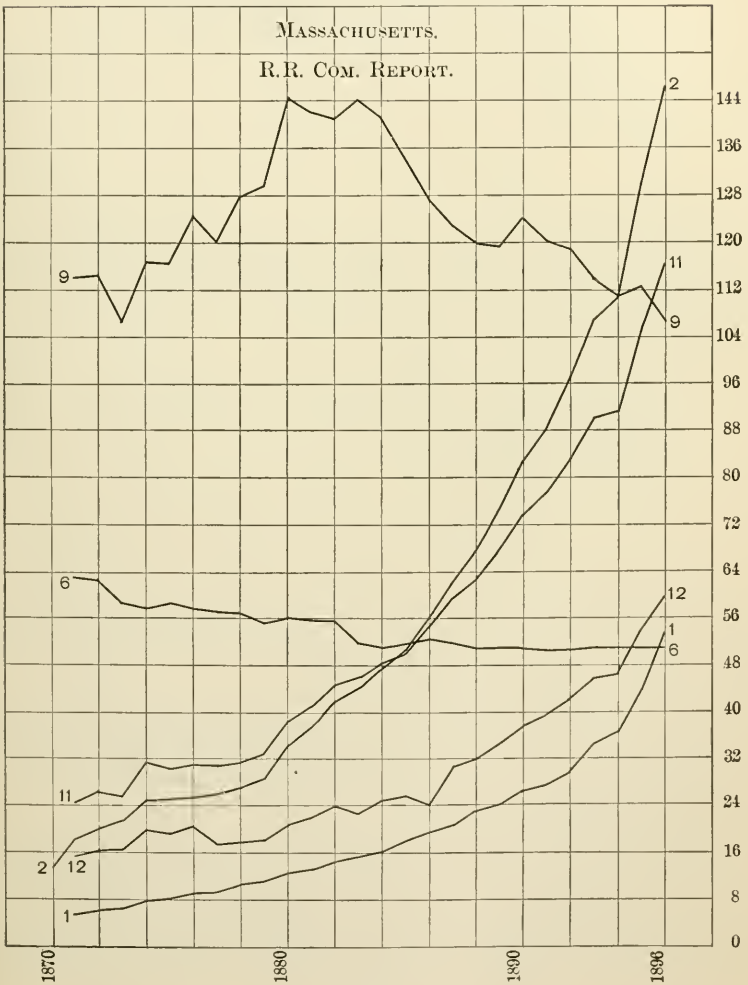

FIG. 6.

Mr. Davis. poles, center-construction (assuming that there would be a double track); trolley wire 423 200 circular mils; poles set every 100 ft., each weighing 1 300 lbs. (30 ft. by 8 ins. and 7 ins. and 6 ins.) set in concrete, would cost for one mile, approximately, $3 237 (one-half of poles with brackets and feeder supports, etc., set at $50 each; trolley 6 748 lbs., at 15 cents; special line material, $500; labor, $400).

To recapitulate: One mile of track equipped electrically will cost:

(11) Rail bonding. In this case the cost of ground-return circuit is not an extra cost against the electric road, for the rails are used, they having sufficient capacity to carry the current. (See Equipment)	$497
(12) Overhead-trolley line	3 237
Total extra per mile for electric trolley railroad	$3 734

If a third-rail system were used, which is probable where the road is on its own right-of-way and where high speeds and frequent service are necessary, together with longer and heavier trains than the single cars of a railway system, then the increased cost would be approximately:

(11) Rail bonding, as before	$497
(12) 70.7 tons 90-lb. third-rail, special section, at $30	2 121
528 treated wooden supports at $1	528
176 joints to bond, at $3	528
Laying, at 5 cents per foot	264
Total extra per mile for third-rail electric railroad	$3 938

The apparent difference in favor of the overhead trolley must not be assumed to exist generally; accidentally, it happens to be less in the first case than in the second, while usually, on long lines with dense traffic, the reverse would be true.

There being no items applicable exclusively to steam railroads, to balance (11) and (12), it is seen that the first cost of construction of an electric road will be greater, and that this increase will amount to several thousand dollars per mile of track, depending upon the conditions existing in each case.

Under (*c*) "Equipment," if the road is to be equipped to operate freight trains by electric power, there are two possible ways of doing so; first, by using the present style of freight cars and hauling them by electric locomotives; or, second, by equipping each car with motors, and thus making each an independent unit. The latter would only be feasible could the units be coupled together and controlled at the

Mr. Davis.

EXPLANATION.

Curve.		Multiplier for Vertical Scale.
1.	Passenger Train-Miles	1 000 000
2.	Total Passengers Carried	1 000 000
3.	Passengers Carried one Mile	10 000 000
4.	Gross Passenger Earnings, in Dollars	1 000 000
5.	Receipts per Passenger per Mile Carried, in Cents	1/10
6.	" " " Carried, in Cents	1
7.	" " " Train-Mile, in Cents	1
8.	" " Mile of Track, in Dollars	100
9.	Number of Passengers Carried per Mile of Track	100
10.	Miles Traveled per Passenger	1/4
11.	Passenger Rides per Capita per Annum	1
12.	Receipts per Capita per Annum, in Dollars	1/10

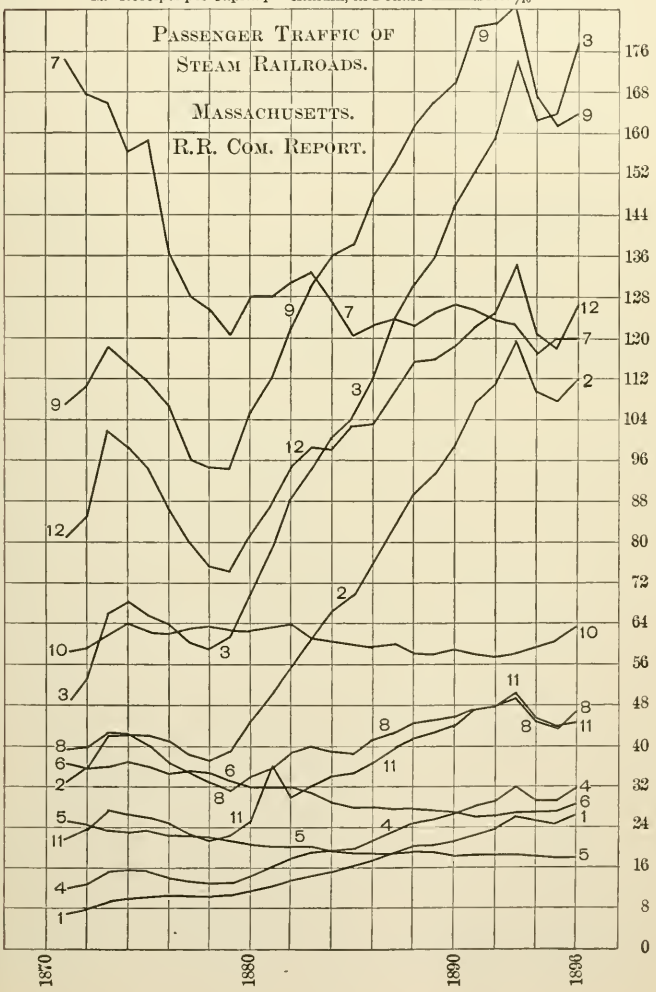

FIG. 7.

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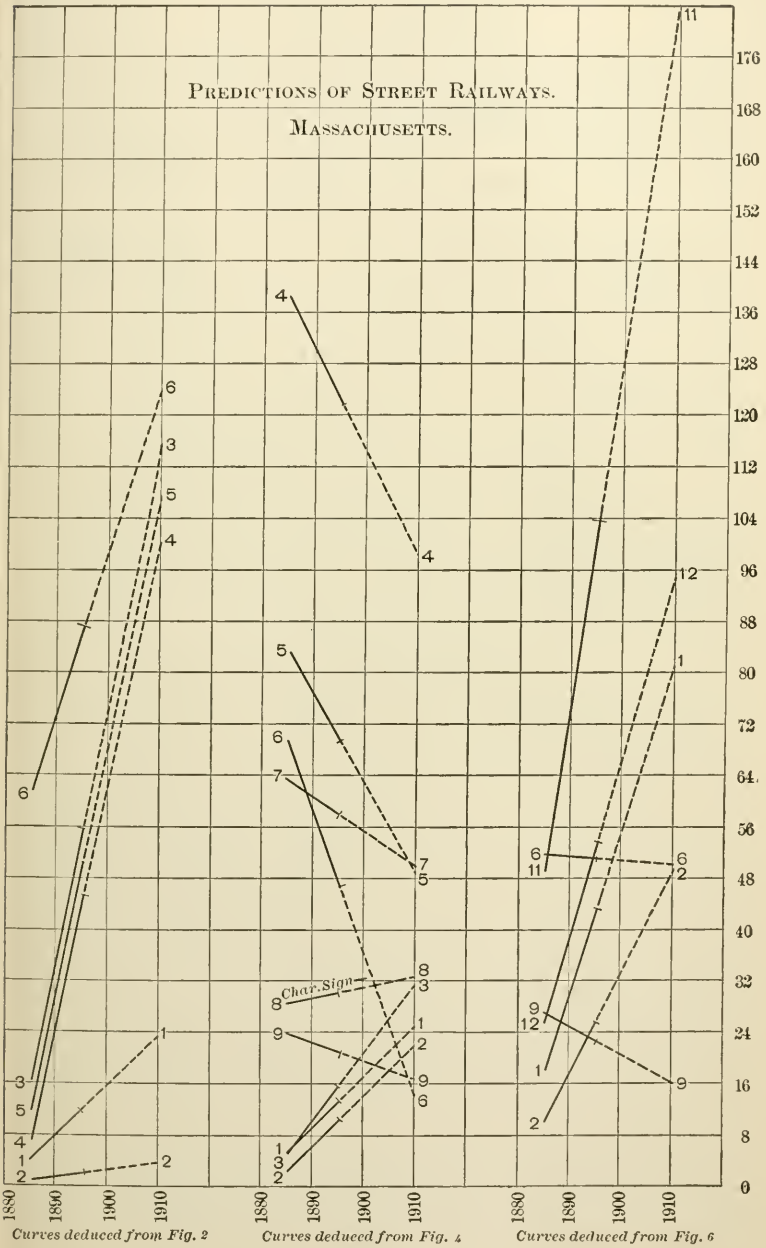
EXPLANATION OF FIG. 8.

Curve.	Multiplier for Vertical Scale.
Curves Deduced from Fig. 2.	
1. Total Miles of Track	100
2. Annual Increase of Mileage	100
3. Total Capital Stock, in Dollars	500 000
4. " Bonded Debt, " "	500 000
5. " Capitalization, " "	1 000 000
6. " " per Mile of Track Owned, in dollars	500
Curves Deduced from Fig. 4.	
1. Gross Traffic Earnings, in Dollars	1 000 000
2. Total Interest on Bonded Debt, in Dollars	100 000
3. " Dividends on Capital Stock, in Dollars	100 000
4. Gross Traffic Earnings per Mile of Railroad, in Dollars	100
5. Percentage of Expenses to Gross Earnings	1
6. " " Interest Paid on Bonded Debt	$\frac{1}{10}$
7. " " Dividends Paid on Capital Stock	$\frac{1}{10}$
8. Gross Earnings per Revenue Train-Mile (Car-Mile) in Cents	1
9. " Expenses " " " " " " " "	1
Curves Deduced from Fig. 6.	
1. Passenger Train-Miles (Car-Miles)	1 000 000
2. Total Passengers Carried	10 000 000
4. Gross Passenger Earnings (Same as Curve 1, Fig. 4)	.....
6. Receipts per Passenger Carried, in Cents	$\frac{1}{10}$
7. " " " Train-Mile (Same as Curve 8, Fig. 4)	.....
8. " " Mile of Railroad (Same as Curve 4, Fig. 4)	.....
9. Number of Passengers Carried per Mile of Railroad	10 000
11. Passenger Rides per Capita per Annum	1
12. Receipts per Capita per Annum, in Dollars	$\frac{1}{10}$

head of the train; but imagine interchanging and making up a train of 50 to 100 cars arranged in this way by any present known mode of electric equipment, control and propulsion; in fact, no arguments seem necessary to show the utter folly of attempting it. The only feasible method is evidently by electric locomotives at the head of such trains. The best practical example of this mode of operation is given in the Baltimore Tunnel by the Baltimore and Ohio Railroad, where 100-ton electric locomotives haul freight and passenger trains, together with their steam locomotives. They do the work satisfactorily, but the writer has seen no claims advanced to show that they are economical; in fact, the only defence for their use is one of cleanliness and ventilation in the tunnel, which is more satisfactory to passenger traffic, and, undoubtedly, attracts and stimulates it. The installation cost, if the writer's memory serves, over $2 000 000 for a few miles of track; so that the interest on this sum is much more than enough to offset any saving in operation, if even the latter can be shown, which is extremely doubtful. Such an investment could not be justified on any grounds if only freight were handled, although it may be by an increase in the passenger traffic. Freight trains are slow-moving, heavy units, requiring large powers constantly applied, and are run at comparatively infrequent intervals. The effort of all steam railroad man-

Mr. Davis.



Curves deduced from Fig. 2 *Curves deduced from Fig. 4* *Curves deduced from Fig. 6*

FIG. 8.

Mr. Davis.

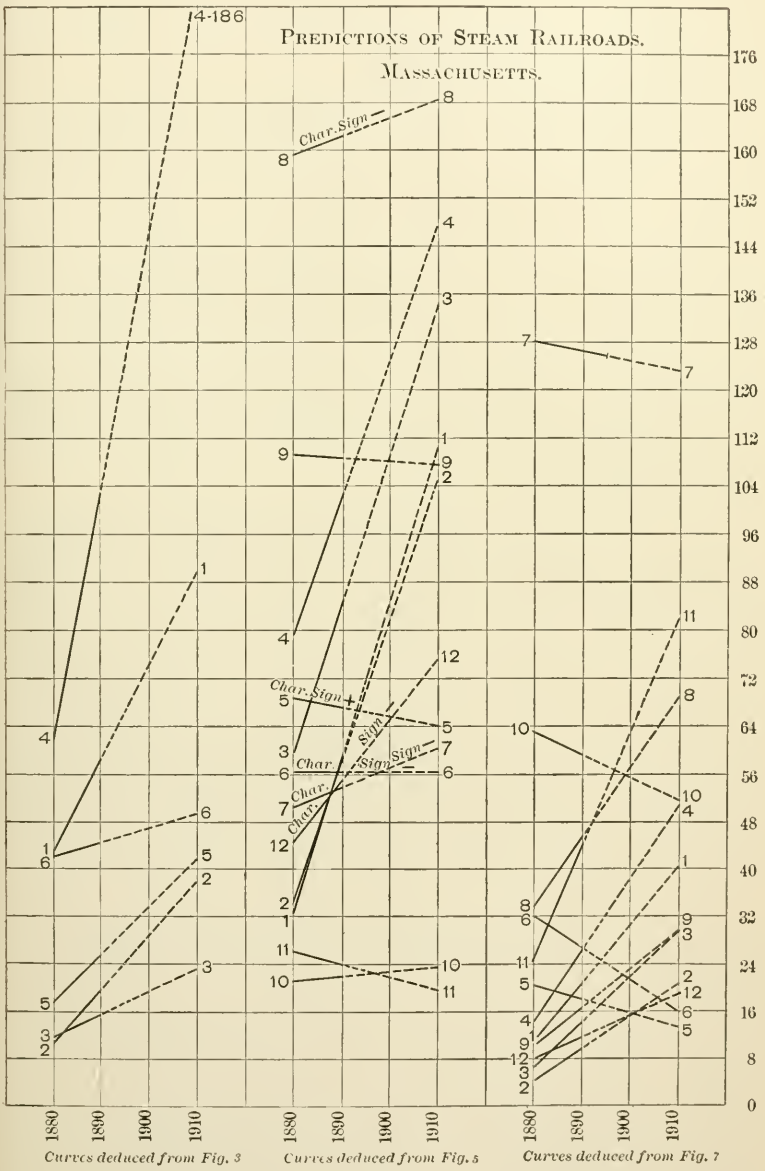
EXPLANATION OF FIG. 9.

Curve.	Curves Deduced from Fig. 3.	Multiplier for Vertical Scale.
1.	Total Miles of Track	100
2.	Annual Increase of Mileage	10
3.	Total Capital Stock, in Dollars	10 000 000
4.	" Bonded Debt, " "	1 000 000
5.	" Capitalization, " "	10 000 000
6.	" " per mile of Railroad, in Dollars	1 000
	Curves Deduced from Fig. 5.	
1.	Gross Traffic Earnings, in Dollars	1 000 000
2.	Total Interest on Bonded Debt, in Dollars	100 000
3.	" Dividends on Capital Stock, in Dollars	100 000
4.	Gross Traffic Earnings per Mile of Railroad, in Dollars	100
5.	Percentage of Expenses to Gross Earnings	1
6.	" " Interest Paid on Bonded Debt	$\frac{1}{10}$
7.	" " Dividends " " Capital Stock	$\frac{1}{10}$
8.	Gross Earnings per Revenue Train-Mile, in Cents	1
9.	" Expenses " " " " "	1
10.	Percentage of Passenger Earnings to Gross Earnings	$\frac{1}{2}$
11.	" " Freight " " " "	$\frac{1}{2}$
12.	" " Interest and Dividends Paid on Capital Stock and Bonded Debt	$\frac{1}{10}$
	Curves Deduced from Fig. 7.	
1.	Passenger Train-Miles	1 000 000
2.	Total Passengers Carried	10 000 000
3.	Passengers Carried One Mile	100 000 000
4.	Gross Passenger Earnings, in Dollars	1 000 000
5.	Receipts per Passenger per Mile Carried, in Cents	$\frac{1}{10}$
6.	" " " Carried, in Cents	1
7.	" " " Train-Mile, in Cents	1
8.	" " Mile of Railroad, in Dollars	100
9.	Number of Passengers Carried per Mile of Railroad	1 000
10.	Miles Traveled per Passenger	$\frac{1}{4}$
11.	Passenger Rides per Capita per Annum	1
12.	Receipts per Capita, per Annum, in Dollars	1

agers is to increase train-loads, so as to decrease expenses per train-mile. If we apply the following argument and figures, changing the example to moving heavy freight trains, it will be seen at once that such traffic cannot now be handled by electric locomotives, involving heavy line work and large central stations (idle most of the 24 hours), for the main reason that interest on first cost will more than offset the largest possible saving in expenses. This saving would not exist, but, if it should, more than 50% would be needed.

Passenger cars would cost the same in either case, as the cost of installing electric lighting and heating systems in the cars would balance the present gas lighting and steam heating systems of railroad passenger cars.

Should motor cars be used, carrying passengers, there would be a saving equal to the cost of one passenger car per train in favor of the electric road. If an electric locomotive were used, there would be no such saving. If every car were a motor car, then the cost of a steam


PREDICTIONS OF STEAM RAILROADS.
MASSACHUSETTS.
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176
168
160
152
144
136
128
120
112
104
96
88
80
72
64
56
48
40
32
24
16
8
0
1880
1890
1900
1910
Curves deduced from Fig. 3
Curves deduced from Fig. 5
Curves deduced from Fig. 7

FIG. 9.

Mr. Davis.

locomotive per train would be saved in the case of electric traction; this, however, would add to the cost of each car. This method is not satisfactorily solved to-day, although short trains are operated with more or less success.

Steam locomotives and tenders are to steam roads what motor cars (or electric locomotives), electric-feeder lines and ground-return circuits, and central power stations (without buildings) are to electric railways. To show the relative costs, let us again consider the former example. The feeder line would be about two 4/0 B. & S. G. wires of bare copper, as the company owns the right-of-way, and crossings of public highways at grade are unlikely on roads of the character we are discussing; these would cost, in place, $1 117 ($1 012 for copper and $105 for stringing). It will be noticed that the trolley wire suggested is much larger than that used in general practice, and, as it has large carrying capacity, part of its cost might have been included in the cost of feeders. If the third rail were used in place of an overhead trolley, as explained before, this feeder system would be unnecessary, as the 90-lb. third rail when properly bonded has enough section to carry the current.

In our example the area of copper in the overhead system is 846 400 circular mils, equivalent to 920 mils diameter, which gives a total cross-section of 0.657 sq. in. A 90-lb. rail has a sectional area of 9 sq. ins. Taking the relative conductivity of steel and copper as 1 to 6, we find that each rail is equivalent to 1.5 sq. ins. of copper, and both rails to 3 sq. ins.; therefore, until the overhead system had a cross-section of more than 3 sq. ins., there would be no necessity for any copper ground-return circuit, so long as the bonding section at the joints was the same as that of the overhead work. In case the area of both rails divided by 6, is less than the area of overhead copper, the copper ground-return wire should be of a section equivalent to the excess in section of overhead copper. This ground return can be run on the pole line and connected down to the rails at intervals, or along the track, as may seem best in any given case.

The central power station of 1 125 nominal H.-P. could be erected complete (without buildings included under "construction") for $50 per H.-P., which would provide a plant of the most modern kind, giving the greatest economy (in the largest sense). In general, it would consist of a steel stack lined with fire-brick, water-tube boilers, pumps, piping, condensers, heaters, blowers, direct-connected Corliss engines, direct-connected generators, slate switchboards, best indicating and recording instruments, etc., with all the best labor-saving and automatic devices which help to make a thoroughly successful plant. The total cost of the power plant would be $56 250. We will not discuss the use of an electric locomotive, but assume the use of motor cars drawing trailers, as it would only make a small difference

in the total amount given in the example, which figures, in any case, are only indicative. The same can be said of equipping each car as a motor car. Mr. Davis.

In the estimate of 200 tons as the weight of a 6-car train, one being a motor car, the car bodies assumed are 45 ft. over sills and 51 ft. over all, with two trucks having four wheels each, the wheels being 33 ins. in diameter; car body weighing 22 000 lbs.; trucks, 13 000 lbs.; total, 35 000 lbs.; average maximum passengers, 150 at 150 lbs. each, 22 500 lbs.; making the total weight of the passenger car loaded 57 500 lbs. The motor car is assumed to have four motors, one on each axle, or geared to it, its total weight loaded being 112 500 lbs. (56.25 tons loaded, 45 tons empty, all on drivers). The trucks of this motor car would be heavier than those of the trail cars, and with larger wheels. With two four-wheel pivotal trucks, the weight on each driving wheel would be 14 082 lbs. loaded, or 11 250 lbs. empty. The trail cars can be assumed to cost, complete, for steam or electricity, about $3 500; the motor car complete, about $7 000. A steam locomotive and tender of the same power and tractive force would cost about $9 000 (engine, $7 700; tender, $1 300). It will be noticed that the heating of cars has had no consideration herein. While this would add to the first cost of an electric system, in increasing the required capacity of transmission line and power house, and would undoubtedly add to the cost of operating as compared with steam-locomotive practice, yet available data are so unreliable that a discussion of them is postponed. To recapitulate: On the basis of our example, we see that items 2, 5 and 6 of the electric system of equipment leave only item 1 on the steam side of the ledger to balance them, and this it comes nowhere near doing. The New York Central Railroad, in 1885, with 2 716 miles of single track, had $2 337 (5.7%) per mile invested in locomotives for passenger and freight service, and must have considerably more to-day. If we deduct investment for freight locomotives and compare with the following, it gives some idea of the difference in first cost of equipment against the electric system with only passenger service considered:

Feeder line	$1 117
Ground return (in this case no additional cost)	.....
Central power station	56 250
	$57 367
Deduct difference between 3 motor cars and 3 locomotives and tenders	6 000
Total extra per mile for electric trolley road	$51 367

To estimate a 10-mile road at ten times the above would be unfair, for, unless the train density remained the same on every mile, and a power station were built for every mile, it could not be in direct pro-

Mr. Davis. portion. In general, the feeder system increases in cost with the distance, while the power station increases in cost with the weights moved and their speed. There would, undoubtedly, be found conditions of very light trains at very frequent intervals over short distances where the larger cost of locomotives and tenders would balance the items which usually make electricity the more costly; these conditions must more nearly approach those which now exist in street railway service. It is evident that with units weighing 10 to 15 tons, run at intervals of 20 seconds to 5 minutes, any steam-locomotive system would prove undesirable, both by its greater first cost and operating expenses.

"Electric feeder lines and ground-return circuit" is placed under the heading "equipment" rather than under "construction" for the purposes of this discussion; in a system operated by electricity the opposite course would be pursued, and has been followed in considering "total expenses." "Central power stations" (all but buildings) would naturally be in the motive power department, and are so considered herein.

"General" items of "first cost" would be unaffected by the character of the system. A few years ago, when the word "electricity," as applied to street railways, was more attractive in banking and investing circles, higher prices for bonds might have been obtained by the use of that system, but elements of this kind are not within the scope of this discussion. Taxes are based on value, not on cost, and are therefore unaffected by any particular system.

Should an existing railroad change its motive power on all or a part of its system, the following items would represent the investment which would be made, the interest upon which must either be saved or earned to warrant it:

(*b*) Construction.

1. Engineering.
(Electric) 11. Rail bonding.
(Electric) 12. Overhead-trolley line or third rail.
15. Buildings for central power stations.

(*c*) Equipment.

(Electric) 2. Motor cars or electric locomotives.
3. Passenger cars (trail cars, or if motors are applied to every car).
(Electric) 5. Electric feeder lines and ground-return circuit.
(Electric) 6. Central power stations (not including buildings which are considered under "construction").

(*d*) General.

1. Discount on bonds.
2. Interest on bonds to opening of road.
3. Taxes to opening of road.
5. Contingent and miscellaneous, not itemized, to opening of road.

TABLE No. 2.—Cost of New York Central and Hudson River Railroad, 1885. Mr. Davis.

(2 716.05 miles of single track.)

Item.	Cost per mile of single track.	Percentage.
1. Grading and masonry	$7 749	18.9
2. Bridges, etc.	1 066	2.6
3. Superstructure	11 439	27.9
4. Stations, etc.	5 453	13.3
5. Land and land damages	5 576	13.6
6. Locomotives	2 337	5.7
7. Passenger cars	574	1.4
8. Freight cars	5 576	13.6
9. Engineering	1 107	2.7
10. Floating equipment	123	0.3
Total	$41 000 (approx.)	100.0
11. Stock	$32 900	
12. Bonds	20 700	
Total capitalization	$53 600	

All other items would be unaffected on an existing system. Taking them up in detail:

Engineering expenses would not be as great as in the case of a new line. Items 11 and 12 under (*b*) "Construction," are applicable to this case, as already discussed for new lines. Buildings for power stations would be an additional cost, unless steam locomotives were entirely discarded, and old round houses utilized; this, however, is hardly likely. Under practically all circumstances steam locomotives and tenders would be utilized on other parts of the system or for freight service, and therefore no allowance need be made for abandoning them. Items 2 and 3 under (*c*) "Equipment" should be new, and old equipment used elsewhere. Any attempt to use existing trucks or car bodies, either for motor cars or trailers, would be as bad a mistake as was made in the early days of the change from horse to electric traction on street railways. Items 5 and 6, under the same heading, are also applicable to this case, as already discussed for new lines. Items 1, 2, 3 and 5, under (*d*) "General" would have to be charged against the change, in proportion to the cost of the same. For all practical purposes, the total cost of a change of system is seen to be about equal to the difference in cost of two new lines, as follows:

(*b*) Construction	$3 734	per mile of track.
(*c*) Equipment	51 367	" "
(*d*) General (15%)	8 265	" "
Total	$63 366	" "

Mr. Davis.

TABLE No. 3.—ELECTRIC SUBURBAN AND

NAME OF RAILWAY.	Index No. of Ry.	Total passengers carried per annum; year ending June 30th, 1898.	Total length of single track operated (miles).	Percentage of operating expenses to gross earnings.	Total passengers carried per mile of single track operated.	Total car-miles operated per annum.
MAINE (19 roads).						
Augusta, Hallowell & Gardiner Railroad	S 1	816 115	7.00	53	116 588	201 534
Bangor, Orono & Old Town Railroad	S 2	1 225 028	17.40	88	73 414	408 570
Biddeford & Saco Ry	S 3	337 860	5.72	67	58 984	136 312
Lewiston & Auburn Horse R. R	S 4	1 160 445	14.00	77	82 888	350 000(a)
Portland & Cape Elizabeth Ry	S 5	1 095 666	11.52	67	90 600	128 945
Portsmouth, Kittery & York St. Ry.	S 6	812 220	15.20	58	54 748	250 000(a)
Rockland, Thomaston & Camden St. Ry	S 7	960 578	16.64	62	57 727	268 609
Totals and averages	7	6 407 912	87.48	74	73 249	1 743 970
CONNECTICUT (31 roads).						
Danbury & Bethel St. Ry	S 8	823 669	10.58	60	77 851	244 533
Derby Street Ry	S 9	712 344	5.89	56	120 958	154 275
Fair Haven & Westville R. R	S 10	4 898 363	19.89	56	246 272	1 283 642
Hartford, Manchester & Rockville Tramway	S 11	599 429	17.88	63	33 525	243 542
Hartford & West Hartford Horse R. R	S 12	375 375	9.70	98	38 698	188 892
Meriden Electric R. R	S 13	1 439 285	17.00	77	84 663	445 864
Norwalk Street Ry	S 14	748 842	7.52	69	99 580	167 592
Norwalk Tramway	S 15	816 113	17.52	78	46 466	302 608
Norwich Street Ry	S 16	1 256 626	11.91	70	105 476	223 347
Torrington & Winchester St. Ry	S 17	436 726	12.56	64	39 548	162 684
Bristol & Plainville Tramway	S 18	543 032	7.34	84	73 978	145 629
Central Railway & Electric (New Britain)	S 19	1 434 105	16.55	75	86 713	338 403
Totals and averages	12	14 143 909	154.34	67	91 641	3 901 011
MASSACHUSETTS (103 roads).						
Arlington & Winchester	S 20	330 594	3.33	67	99 277	79 210
Athol & Orange	S 21	602 227	6.88	63	87 533	114 075
Braintree & Weymouth	S 22	992 063	11.88	65	83 507	267 335
Bridgewater, Whitman & Rockland	S 23	546 485	12.68	75	43 098	220 363
Brockton, Bridgewater & Taunton	S 24	1 353 082	21.07	58	64 260	490 006
Brockton & East Bridgewater	S 25	364 512	9.22	75	39 531	105 528
Dartmouth & Westport	S 26	551 674	13.73	70	40 163	337 232
Dighton, Somerset & Swansea	S 27	775 266	16.78	71	46 188	282 591
Fitchburg & Leominster	S 28	2 178 863	21.43	64	101 673	671 274
Gloucester, Essex & Beverly	S 29	1 556 243	22.71	61	50 911	355 932
Greenfield & Turners Falls	S 30	712 647	12.86	64	55 416	178 076
Haverhill & Amesbury	S 31	1 408 722	26.04	65	54 098	433 833
Haverhill, Georgetown & Danvers	S 32	477 268	6.13	75	77 858	143 160
Hingham	S 33	907 048	19.38	74	46 801	291 842
Hoosac Valley (North Adams)	S 34	1 536 505	13.00	71	118 138	349 752
Interstate Consolidated (R. I.)	S 35	2 589 797	22.42	68	115 513	660 306
Leominster & Clinton	S 36	856 817	11.46	58	74 733	248 306
Lowell, Lawrence & Haverhill	S 37	8 981 702	64.82	56	138 572	1 741 291
Lowell & Suburban	S 38	7 679 147	64.77	61	118 545	1 938 352
Lynn & Boston	S 39	29 063 234	153.83	57	188 928	5 800 287
Marlborough	S 40	756 465	7.54	90	100 314	223 834

(a) Estimated.

INTERURBAN RAILWAYS (R. R. COM. REPORT). Mr. Davis.

PASSENGER EARNINGS.								
Per mile of single track operated.	Per car-mile operated (cents).	Per passenger per trip (cents).	Total per annum from passengers; year ending June 30th, 1898.	No. of passengers carried per car-mile operated.	Percentage of dividends paid; year ending June 30th, 1898.	Permanent investment per mile of single track owned; year ending June 30th, 1898.	Capital stock and debts per mile of single track owned; year ending June 30th, 1898.	Total operating expenses per car-mile operated (cents).
$5 802	20.15	4.97	$40 227	4.04	4.0	$26 982	$39 687	10.90
3 659	15.58	5.19	61 251	2.99	8.0	15 208	16 475	13.76
4 363	18.30	7.38	24 837	2.47	....	23 483	27 651	12.27
3 721	14.80(*a*)	4.48	51 815	3.31 (*a*)	....	29 605	31 985	11.51 (*a*)
3 795	43.31	5.09	54 782	8.49	1.0	52 977	55 637	28.81
2 736	16.42(*a*)	5.03	41 061	3.25 (*a*)	1.2	26 556	26 788	9.57 (*a*)
4 099	25.39	7.10	48 028	3.02	....	19 058	31 631	15.93
$3 680	18.46	5.02	$322 001	3.10	....	$28 839	$25 409	13.71
4 045	17.51	5.13	42 826	3.37	4.0	50 844	47 399	10.64
6 024	23.00	4.98	35 486	4.62	....	55 515	58 527	13.02
12 459	19.31	5.06	247 812	3.82	6.0	36 436	43 298	10.87
3 422	25.13	10.20	61 192	2.46	....	29 317	28 002	15.92
3 051	15.67	7.88	29 599	1.99	....	58 323	52 046	15.49
4 245	16.19	5.08	72 176	3.23	....	94 373	92 507	12.63
4 844	21.74	4.86	36 430	4.47	4.5	31 338	32 159	15.09
2 099	12.16	4 50	36 790	2.70	2.0	38 927	33 466	9.58
4 529	24.17	4.29	53 973	5.63	....	45 498	42 511	17.08
1 927	14.88	4.86	24 202	3.05	....	29 987	28 933	9.57
3 641	18.34	4.90	26 718	3.73	3.0	35 827(*b*)	31 460(*b*)	15.49
3 856	18.86	4.45	63 821	4.24	....	78 934(*b*)	65 123(*b*)	14.26
$4 736	16.17	5.16	$731 025	3.62	....	$49 553	$50 229	12.53
4 938	20.76	4.97	16 444	4.17	....	29 831	28 928	13.86
4 281	25.82	4.89	29 339	5.27	8.0	19 771	17 464	16.21
4 087	18.16	4.89	48 258	3.71	3.0	21 248	20 691	11.81
2 136	12.29	4.96	26 960	2.47	....	16 577	16 940	9.26
3 381	14.55	5.27	69 484	2.76	3.0	24 903	24 405	8.29
1 988	17.46	5.06	18 352	3.45	....	20 981	20 341	13.11
7 055	28.74	17.57	95 578	1.63	8.0	26 594	24 541	20.18
3 668	21.79	7.94	61 516	2.75	....	27 541	27 328	15.56
6 267	20.01	6.16	134 014	3.24	8.0	25 905	25 787	12.84
2 599	16.59	5.11	58 698	3.24	3.0	17 837	17 880	10.23
2 689	19.38	4.85	33 677	4.00	4.0	16 835	16 386	12.37
3 443	20.67	6.36	88 060	3.24	....	25 993	25 811	13.46
3 914	16.76	5.03	23 757	3.33	....	18 176	17 096	12.63
2 304	15.31	4.92	44 197	3.75	....	22 059	22 741	11.30
5 894	21.92	4.99	75 214	4.38	6.0	19 558	17 881	15.64
6 039	20.50	5.23	134 774	3 99	6.0	20 465	25 083	14.02
805	17.57	5.09	43 165	3.45	....	31 282	32 487	10.23
6 980	25.98	5.04	448 938	5.15	4.0	48 795	47 401	14.68
5 986	20.00	5.05	380 177	3.96	3.0	36 004	36 169	12.28
9 648	25.59	5.11	1 472 154	5.01	8.0	58 851	59 200	14.75
5 117	17.24	5.10	35 952	3.38	....	24 720	26 967	15.60

(*b*) Includes cost of electric lighting plant, which could not be separated.

Mr. Davis.

TABLE No. 3 (*Concluded*).—ELECTRIC SUBURBAN AND

NAME OF RAILWAY.	Index No. of Ry.	Total passengers carried per annum; year ending June 30th, 1898.	Total length of single track operated (miles).	Percentage of operating expenses to gross earnings.	Total passengers carried per mile of single track operated.	Total car-miles operated per annum.
Milford, Holliston & Framingham	S 41	1 668 831	20.31	60	82 051	465 991
Natick & Cochituate	S 42	1 089 877	11.00	84	99 080	257 772
Newburyport & Amesbury	S 43	1 139 652	18.38	92	62 015	320 056
Newton	S 44	1 827 120	15.78	67	115 743	465 458
Newton & Boston	S 45	1 098 486	13.82	71	79 468	456 530
Norfolk Central (Dedham)	S 46	712 551	6.52	72	109 337	185 583
Norfolk Suburban (Hyde Park)	S 47	1 972 521	10.95	77	180 188	423 986
Northampton	S 48	1 812 627	17.12	62	105 847	487 369
North Woburn	S 49	809 869	7.80	70	103 803	189 073
Providence & Taunton	S 50	673 714	14.92	36	45 149	99 983
Quincy & Boston	S 51	2 712 342	30.06	64	90 240	564 141
Reading & Lowell	S 52	332 530	11.84	93	28 085	146 373
Rockland & Abington	S 53	1 521 581	17.22	83	88 336	397 812
Rockport	S 54	567 164	8.16	69	69 505	146 377
Southbridge & Sturbridge	S 55	552 586	7.75	68	71 320	149 588
South Middlesex (Natick)	S 56	1 134 670	13.07	63	86 815	269 181
Taunton & Brockton	S 57	949 244	17.23	76	55 093	249 362
Wakefield & Stoneham	S 58	1 129 681	15.30	58	73 835	374 431
Warren, Brookfield & Spencer	S 59	966 236	19.22	61	50 275	288 827
Worcester & Blackstone Valley	S 60	281 043	6.92	82	40 665	110 098
Worcester & Marlborough	S 61	1 229 738	17.36	65	70 837	342 877
Worcester & Suburban	S 62	3 275 611	21.91	68	149 483	714 529
Totals and averages	43	91 288 034	864.63	64	105 567	22 037 982
			(c) *Totals and Averages of Table No. 3.*			
Maine	(d) 7	6 407 912	87.48	74	73 249	1 743 970
Connecticut	(d)12	14 143 909	154.34	67	91 641	3 901 011
Massachusetts	(d)43	91 288 034	864.63	64	105 567	22 037 982
Totals and averages	(d)62	111 839 855	1106.45	65	101 079	27 682 963

(c) Railways in Rhode Island, New Hampshire and Vermont are not treated in this table.

It should be remembered that the figures at the foot of page 875 are not exact, nor would it be fair to multiply by the number of miles of track equipped.

It is evident that, whether a new line be built under the same conditions of existing steam railroads, or an old system be changed over, the first cost will be against electric motive power; and to overcome this disadvantage the operating expenses must be reduced sufficiently to more than take care of the interest on this additional cost; or else the change must create an increased traffic which justifies it.

Wellington* gives a table showing the distribution of first cost of the New York Central and Hudson River Railroad in 1885, from which Table No. 2 has been calculated.

* "Economic Theory of Railway Location," p. 71.

INTERURBAN RAILWAYS (R. R. COM. REPORT). Mr. Davis.

PASSENGER EARNINGS.								
Per mile of single track operated.	Per car-mile operated (cents).	Per passenger per trip (cents).	Total per annum from passengers; year ending June 30th, 1898.	No. of passengers carried per car-mile operated.	Percentage of dividends paid; year ending June 30th, 1898.	Permanent investment per mile of single track owned; year ending June 30th, 1898.	Capital stock and debts per mile of single track owned; year ending June 30th, 1898.	Total operating expenses per car-mile operated (cents).
$4 103	17.91	5.00	$83 283	3.58	....	$20 096	$19 351	10.73
5 044	21.53	5.09	55 070	4.22	6.0	11 463	10 580	18.21
3 259	18.71	5.26	56 982	3.56	....	29 847	31 166	17.16
6 069	20.58	5.24	91 465	3.92	8.0	33 830	32 239	13.83
4 729	14.32	5.95	54 322	2.40	5.0	40 179	39 545	10.20
5 437	19.09	4.97	35 351	3.88	....	26 919	19 487	13.81
9 079	23.44	5.04	98 998	4.65	7.0	25 322	24 939	18.05
5 391	18.94	5.09	89 466	3.71	8.0	22 965	21 778	11.82
5 234	21.60	5.04	40 690	4.28	....	26 302	27 622	15.13
2 382	35.56	5.28	35 456	6.73	....	30 302	28 821	12.86
4 515	24.06	5.00	128 857	4.80	3.5	22 225	20 806	15.53
1 413	11.44	5.03	16 588	2.27	....	13 074	13 685	10.63
4 521	19.58	5.12	75 315	3.80	6.0	16 103	15 937	16.37
3 484	19.42	5.01	28 358	3.87	6.0	16 888	15 602	13.52
3 595	18.63	5.04	27 177	3.02	5.0	16 116	15 842	12.74
4 370	21.22	5.03	56 517	4.21	6.0	19 797	19 189	13.39
2 824	19.51	5.13	48 510	3.80	6.0	16 323	16 149	14.78
4 056	16.58	5.49	61 838	3.01	5.0	25 291	25 163	9.59
2 510	16.17	4.99	48 041	3.34	....	17 662	17 204	10.23
2 033	12.78	5.00	14 070	2.55	....	12 867	12 671	10.44
3 780	19.14	5.34	64 787	3.58	6.0	26 690	25 453	12.48
7 044	21.60	4.71	152 402	4.58	4.0	37 364	38 688	14.63
$5 437	21.33	5.15	$4 702 191	4.14	....	$30 131	$30 507	13.74
Electric Suburban and Interurban Railways.								
$3 680	18.46	5.02	$322 001	3.10	....	$28 839	$25 409	13.71
4 736	16.17	5.16	731 025	3.62	....	49 553	50 229	12.53
5 437	21.33	5.15	4 702 191	4.14	....	30 131	30 507	13.74
$5 201	20.79	5.14	$5 755 217	4.04		$32 738	$32,855	13.20

(*d*) No. of roads taken in each State.

From Table No. 3 we find that the average cost per mile of single track of 62 suburban and interurban electric railways in New England is $32 738, while the highest is $58 851, and the lowest $11 463. Obviously, it would be unfair to compare the steam railroad chosen with the foregoing average, as the former must exceed the average of its own class by over 25 per cent. Adding this percentage to the foregoing average gives approximately $41 000 per mile of single track as a fair comparative cost for an electric interurban railway with the same standard of construction and equipment as the New York Central and Hudson River Railroad. The construction account of such railways is about 50% of the whole; the equipment 30% (including power stations); and lands, buildings and other permanent investment about 20 per cent. From the foregoing, Table No. 8 has been computed.

Mr. Davis.

TABLE No. 4.—ELECTRIC CITY

NAME OF RAILWAY.	Index No. of Ry.	Total passengers carried per annum; year ending June 30th, 1898	Total length of single track operated (miles).	Percentage of operating expenses to gross earnings.	Total passengers carried per mile of single track operated.	Total car-miles operated per annum.
MAINE (13 roads).						
Bangor St. Ry.	C 1	1 523 837	9.70	82	157 096	326 197
Bath St. Ry.	C 2	451 907	4.25	66	106 331	103 395
Calais St. Ry.	C 3	485 613	7.00	79	69 373	183 960
Portland Railroad.	C 4	5 444 897	30.95	68	175 925	1 155 854
Totals and averages.	4	7 906 254	51.90	72	152 336	1 769 406
CONNECTICUT (31 roads).						
Bridgeport Traction.	C 5	4 108 260	52.60	51	78 107	1 454 638
Hartford Street Ry.	C 6	8 886 229	56.64	69	156 889	2 228 932
Middletown St. Ry.	C 7	101 948	7.30	73	55 061	102 801
New Haven St. Ry.	C 8	3 093 458	26.80	60	115 427	883 731
New London St. Ry.	C 9	666 844	6.96	74	95 810	137 841
Stamford St. Ry.	C 10	618 845	11.05	97	56 004	226 263
Winchester Ave. (New Haven).	C 11	4 381 462	17.54	61	249 798	764 801
Waterbury Traction.	C 12	2 515 116	12.18	58	206 495	435 431
Totals and averages.	8	24 672 162	191.07	63	129 131	6 234 438
MASSACHUSETTS (103 roads).						
Boston Elevated.	C 13	181 321 295	278.35	71	651 410	32 209 150
Braintree.	C 14	944 064	14.26	80	66 204	267 364
Brockton.	C 15	6 787 425	43.37	63	156 511	1 463 110
Commonwealth Ave. (Newton).	C 16	1 447 822	12.73	92	113 724	478 017
Framingham Union.	C 17	636 673	6.36	60	100 137	111 966
Gardner Electric.	C 18	341 038	4.34	82	76 569	77 126
Globe (Fall River).	C 19	6 627 700	29.09	62	227 842	1 280 612
Gloucester.	C 20	1 369 961	11.25	67	121 774	283 601
Holyoke.	C 21	4 233 900	30.56	67	138 535	1 107 962
Pittsfield Electric.	C 22	1 318 035	10.10	65	130 537	260 900
Plymouth & Kingston.	C 23	691 048	8.75	67	78 977	181 650
Springfield.	C 24	11 611 232	61.60	74	188 479	3 133 475
Taunton.	C 25	1 269 894	17.13	54	74 115	383 711
Union (New Bedford).	C 26	3 776 878	19.96	64	189 222	799 738
Wellesley & Boston.	C 27	709 887	4.66	67	152 336	220 980
West Roxbury & Roslindale.	C 28	1 460 920	10.26	66	142 307	323 023
Worcester Consolidated.	C 29	10 637 221	41.81	73	254 388	2 121 402
Woronoco.	C 30	485 636	7.08	92	68 554	257 497
Totals and averages.	18	235 670 539	611.66	72	385 296	44 961 284
Totals and averages (omitting C 13)	17	54 349 244	333.31	71	163 059	12 752 134
NEW HAMPSHIRE (5 roads).						
Union St. Ry.	C 31	592 567	7.50	96	79 008	178 740
Nashua St. Ry.	C 32	954 333	14.46	80	65 928	400 353
Concord St. Ry.	C 33	971 160	12.50	92	77 692	300 000 (*a*)
Manchester St. Ry.	C 34	3 290 386	17.02	68	201 124	762 635
Laconia St. Ry.	C 35	229 171	3.63	75	68 043	59 665
Totals and averages.	5	6 037 617	55.11	77	109 555	1 701 393
		(*b*) *Totals and Averages of Table No. 4*				
Maine.	(*c*) 4	7 906 254	51.90	72	152 336	1 769 406
Connecticut.	(*c*) 8	24 672 162	191.07	63	129 131	6 234 438
Massachusetts.	(*c*) 17	54 349 244	333.31	71	163 059	12 752 134
New Hampshire.	(*c*) 5	6 037 617	55.11	77	109 555	1 701 393
Totals and averages.	(*c*)34	92 965 277	631.39	69	147 237	22 457 371

(*a*) Estimated. Given in report as "96 000 estimated," which is obviously wrong.
(*c*) No. of roads taken in each State.

RAILWAYS (R. R. COM. REPORTS). Mr. Davis.

PASSENGER EARNINGS.								
Per mile of single track operated.	Per car-mile operated (cents).	Per passenger per trip (cents).	Total per annum from passengers; year ending June 30, 1898.	No. of passengers carried per car-mile operated.	Percentage of dividends paid; year ending June 30, 1898.	Permanent investment per mile of single track owned; year ending June 30, 1898.	Capital stock and debts per mile of single track owned; year ending June 30, 1898.	Total operating expenses per car-mile operated (cents).
$6 731	20.00	4.28	$61 932	4.67	....	$58 864	$46 974	16.60
5 063	20.56	4.70	21 190	4.37	2.5	40 592	40 976	13.66
3 221	12.63	4.78	22 964	2.64	...	28 571	30 097	10.06
9 761	26.13	5.51	300 237	4.71	6.0	34 712	34 263	17.95
$7 825	22.97	5.15	$406 323	4.46	....	$35 441	$34 016	16.63
4 581	16.56	5.86	241 007	2.82	....	73 998	74 349	8.56
7 734	19.65	4.92	438 069	3.99	3.5	46 913	41 569	13.60
2 520	17.90	4.57	18 402	3.91	....	32 420	35 943	13.18
5 401	16.38	4.67	144 755	3.50	....	70 898	65 992	9.93
4 454	22.49	4.64	31 003	4.84	5.0	54 963	52 986	16.83
2 726	13.31	4.86	30 122	2.74	....	27 841	27 899	12.95
9 388	21.53	3.76	164 666	5.73	8.16	57 452	40 766	13.30
9 644	26.98	4.67	117 469	5.78	3.0	75 016	87 857	15.59
$6 204	19.01	4.80	$1 185 493	3.95	....	$59 129	$62 517	12.04
32 976	28.50	5.06	8 967 587	5.62	2.25	98 600	103 655	20.39
3 163	16.87	4.78	44 635	3.53	4.0	13 115	13 198	13.46
7 749	22.97	4.95	330 559	4.63	6.0	34 997	34 326	14.52
5 783	15.40	5.08	71 331	3.02	....	29 068	29 145	14.27
4 785	27.17	4.78	29 954	5.68	....	22 523	21 962	16.48
3 940	22.18	5.01	17 020	4.42	....	17 533	18 403	18.32
10 943	24.86	4.80	301 238	5.17	....	75 447	74 886	15 57
6 272	24.88	5.15	68 498	4.83	6.0	33 377	31 211	16.71
7 496	20.68	5.41	227 165	3.82	8.0	29 964	27 950	13.95
6 543	25.32	5.01	65 845	5.05	6.0	13 663	12 451	16.45
4 209	20.28	5.01	34 106	3.25	6.0	18 865	18 577	13.62
9 464	18.60	5.02	577 800	3.70	8.0	35 591	31 362	13.73
4 112	18.36	5.55	63 490	3.30	....	54 556	55 073	9.96
10 327	25.78	5.45	187 728	4.72	6.0	47 899	45 415	16.50
8 449	17.82	5.55	37 609	3.21	8.0	24 759	23 895	11.93
6 568	20.87	4.62	67 188	4.52	3.5	35 877	36 846	13.70
12 863	25.35	5.06	530 259	5.01	8.0	47 803	44 675	18.58
3 243	8.92	4.73	22 933	1.88	2.5	16 273	16 967	18.17
$19 038	25.89	4.94	$11 644 945	5.24	....	$63 794	$61 928	18.84
$8 002	20.99	4.92	$2 677 358	4.26	....	$37 854	$34 379	14.93
4 064	17.05	5.14	30 485	3.31	....	.....	33 333	16.43
3 709	13.39	5.62	53 640	2.38	....	27 181	27 662	10.75
3 969	16.54	5.10	49 626	3.24	6.0	.....	17 480	15.35
8 182	18.26	4.23	139 266	4.31	10.0	18 192	20 564	11.96
3 439	20.92	5.44	12 483	3.85	....	.....	13 774	15.83
$5 180	16.77	4.73	$285 500	3.55	....	$22 321	$24 832	12.89
Electric City Railways (C 13 omitted).								
$7 825	22.97	5.15	$406 323	4.46	....	$35 441	$34 016	16.63
6 204	19.01	4.80	1 185 493	3.95	....	59 129	62 517	12.04
8 002	20.99	4.92	2 677 358	4.26	....	37 854	34 379	14.93
5 180	16.77	4.73	285 500	3.55	....	22 321	24 832	12.89
$72 13	20.28	4.89	$4 554 674	4.13	....	$42 579	$42 031	14.11

(*b*) Railways in Rhode Island and Vermont are not treated in this table.

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TABLE No. 5.—All Electric Railways

	No. of roads in each State.	Total passengers carried per annum; year ending June 30th, 1898.	Total length of single track operated. Miles.	Percentage of operating expenses to gross earnings.	Total passengers carried per mile of single track operated.
Maine	19	14 651 165	182.80	71	79 054
Connecticut	31	39 272 306	387.01	65	101 478
Massachusetts	103	330 889 629	1 590.95	69	207 982
Rhode Island	9	39 979 695	154.88	60	258 133
New Hampshire (1896)	5	6 037 617	51.11	77	109 555
Vermont (1896)	5	1 637 262	27.14	..	.....
Totals and averages	172	432 467 674	2 393.89	67	182 726

TABLE No. 6.—Totals and Averages of Table No. 3, Electric Electric City Rail-

Maine	11	14 314 166	139.38	73	102 698
Connecticut	20	38 816 071	345.41	64	112 376
Massachusetts	60	145 637 278	1 197.94	66	121 572
Rhode Island	....	.....	.....	..	.....
New Hampshire	5	6 037 617	55.11	77	109 555
Vermont	....	.....	.....	..	.....
Totals and averages	96	204 805 132	1 737.84	67	117 890

TABLE No. 7.—Totals and

Table No. 3.—Electric Suburban and Interurban Railways (selected from total number in New England)	†62	111 839 855	1 106.45	65	101 079
Table No. 4.*—Electric City Railways (selected from total number in New England)	†34	92 965 277	631.39	69	147 237
Table No. 5.—All Electric Railways in New England (total number in New England making reports to R. R. Coms.)	†172	432 467 674	2 393.89	67	182 726
Table No. 6.*—Electric City, Suburban and Interurban Railways (selected from total number in New England)	†96	204 805 132	1 737.84	67	117 890

† No. of roads taken.

IN NEW ENGLAND (R. R. COM. REPORTS).*

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Total car-miles operated per annum.	Passenger Earnings.				No. of passengers carried per car-mile operated.	Percentage of dividends paid, year ending June 30, 1898.	Permanent investment per mile of single track owned; year ending June 30, 1898.	Capital stock and debts per mile of single track owned; year ending June 30, 1898.	Total operating expenses per car-mile operated. Cents.
	Per mile of single track operated.	Per car-mile operated. Cents.	Per passenger per trip. Cents.	Total per annum from passengers; year ending June 30, 1898.					
3 273 932	4 720	26.35	5.89	862 885	4.47	.....	29 331	32 263	18.18
10 323 464	5 253	19.45	5.14	2 018 986	3.80	.....	51 177	51 610	12.71
68 206 418	10 632	24.80	5.11	16 545 554	4.85	.....	43 229	44 958	17.11
.....	13 092	.....	5.07	2 027 693	.....	.....	.....	75 698	.....
1 701 393	5 180	16.77	4 73	285 500	3.55	.....	22 321	24 832	12.89
.....	.....	.....	.....	.....	.....	.....	20 362	30 033	.....
83 505 207	9 186	23.60	5.04	21 740 618	4.68	.....	42 717	46 453	16.52

SUBURBAN AND INTERURBAN ELECTRIC RAILWAYS, AND TABLE NO. 4, WAYS (*C*13 OMITTED).

3 513 376	5 225	20.73	5.08	728 324	4.07	.....	31 297	28 614	15.14
10 135 449	5 548	18.91	4.93	1 916 518	3.83	.....	54 850	57 026	12.23
34 790 116	6 160	21.21	5.06	7 379 549	4.18	.....	32 196	31 585	14.18
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1 701 393	5 180	16.77	4.73	285 500	3.55	.....	22 321	24 832	12.89
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
50 140 334	5 932	20.56	5.03	10 309 891	4.08	.....	36 313	36 189	13.91

AVERAGES, TABLES NOS. 3, 4 AND 5.

27 682 963	5 201	20.79	5.14	5 755 217	4.04	.....	32 738	32 855	13.20
22 457 371	7 213	20.28	4.89	4 554 674	4.13	.....	42 579	42 031	14.11
83 505 207	9 186	23.60	5.04	21 740 618	4.68	.....	42 717	46 453	16.52
50 140 334	5 932	20.56	5.03	10 309 891	4.08	.....	36 313	36 189	13.91

* *C* 13 omitted.

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TABLE No. 8.

Item.	N. Y. C. & H. R. R. R.		Electric Railway (Approximate).	
	Per mile of single track.	Per-centage.	Per mile of single track.	Per-centage.
1. Grading and masonry	7 749	18.9	2 050	5.0
2. Bridges, etc.	1 066	2.6	410	1.0
3. Superstructure	11 439	27.9	9 020	22.0
9*a*. Engineering ($\frac{1}{2}$)	554	1.35	410	1.0
A. Rail bonding	.....	.....	410	1.0
B. Overhead electric line	.....	.....	2 460	6.00
C. Paving	.....	.....	5 740	14.0
Total construction	20 808	50.75	20 500	50.0
4. Stations, etc.	5 453	13.3	.....	.....
5. Land and land damages	5 576	13.6	.....	.....
Total land and buildings	11 029	26.9	8 200	20.0
6. Locomotives	2 337	5.7	.....	.....
7. Passenger cars	574	1.4	8 200	20.0
8. Freight cars	5 576	13.6	.....	.....
9*b*. Engineering ($\frac{1}{2}$)	553	1.35	.....	.....
10. Floating equipment	123	0.3	.....	.....
D. Electric feeder line and ground return	.....	.....	4 100	10.00
E. Central power plants	.....	.....	4 100	10.00
Total equipment	9 163	22.35	12 300	30.00

From Table No. 8, we see that items *A*, *B*, 7, *D* and *E* constitute those which would be an increase in the cost of a new electric line under steam-railroad conditions, or to change an existing steam railroad; all other items have already been shown to be the same, although varying, as the table shows, in the case of a street railway. We therefore get an approximate general figure of $20 000 per mile of single track as the increased cost of the electric railroad. This figure should be treated with caution, in any individual case, for it might be exceeded by more than 100% under some circumstances, although it is approximately accurate as an average.

In either a new or an old line there is a limiting factor which sometimes precludes the possibility of using electric traction—the transmission of electric power from the station to the motor. Some problems show at once that, owing to the long distance and infrequent trains of great weight, electric transmission of power would be entirely too costly. Just where the line occurs is hard to define, as it depends upon distance, amount of power, frequency of trains, and their weight. A few years ago it was considered that about ten miles' radius was the limit for one power house; this was before the days of "boosters" and multiphase currents. The development in multiphase transmission has led many to believe that this would make it

possible to operate trunk railroad lines by electric motors successfully. The basis for this is the reduction of line cost by the use of high potentials and the utilization of cheap sources of power such as waterfalls; but it must be remembered that the problem is not solely a question of the original cost of power, but rather the concentration of power and its long-distance transmission coupled with a cheap source, *versus* operation by small isolated units—steam locomotives. Mr. Davis.

In the operation of railways, whether electric or steam lines, one requisite is that the speed of trains must be controlled by the operator; the variations of speed must be numerous, and the change from one to another must be sufficiently smooth to cause no discomfort to the passengers or excessive strain on any of the apparatus. The electric railway motor operating by direct current has long been used for this work with satisfactory results, and in its latest forms is sufficient and serviceable under the rough usage to which it must necessarily be subjected.

With alternating-current motors the regulation of speed is more difficult. Until the recent development in "induction" motors, the so-called "synchronous" motor was the only type worthy of practical consideration. This type derived its name from the fact that of necessity it ran at the same speed as the generator which supplied it with current. Obviously, therefore, the speed could not be varied. Besides this, the "synchronous" motor would not start by itself under load. Hence it was unsuited for operating railway cars. The other form of alternating motor—the "induction" motor—is capable of speed-regulation to a limited extent, and is continually being improved, so that the future may see developments and improvements beyond our most sanguine expectations. Alternating-current "induction" motors have certain characteristics which would be of great advantage in railway work. The most important of these is the absence of the commutator. In other words, there is no electrical connection between the stationary parts of the "induction" motor and the armature or rotating part, and, therefore, there is no exposed metal carrying current, and no sparking due to the use of brushes.

The use of multiphase currents has the following advantages:

(1) They allow some control of the speed of motors.
(2) They provide for starting motors when loaded, but not when over-loaded to the same extent as direct-current motors.
(3) They facilitate insulation.
(4) They overcome some of the effects of induction.
(5) They reduce the cost of conductors.

In short, then, the "induction" motor, like the "synchronous" motor, presents the difficulties of starting under loads as well as speed-control. The two-phase and three-phase "induction" motors have

Mr. Davis. overcome these difficulties to a certain extent, and they offer improvements over direct-current motors.

(1) They require less attention.
(2) They give less trouble from dirt, dust, or water.
(3) They have no commutator.
(4) They have no moving electrical contacts (except in special cases).
(5) There is less danger of burning out armatures.
(6) They require renewal of parts less frequently.

The disadvantages in their use can be summarized thus:

(1) They do not start readily with over-loads.
(2) Their speed cannot be controlled readily.
(3) Their efficiency is lower.

At present the disadvantages outweigh the advantages, in so far as the conditions of railway work are concerned.

It seems, therefore, that the direct application of alternating-current motors to railroad systems is not feasible to-day from a practical standpoint; mainly for the reason that they do not start under heavy over-loads, and they are not easily varied in speed over wide ranges.

Irrespective of the motors, however, the alternating current can be used indirectly in operating railway lines. The method, in brief, is to carry the alternating current from the power station to a point in close proximity to a section of the track. Here the current is changed from alternating to direct by means of the so-called rotary transformer, and then applied as direct current to the cars using direct-current motors, with the usual devices for controlling the speed. This method of using the alternating current introduces an extra transformation, with its concomitant loss, and its use is confined to cases where it is desired to transmit power from a distance. Alternating currents are better adapted than direct currents for transmitting power over long distances, because, with the use of the latter, we are now limited to comparatively low pressures, either in the generating plant (1 000 to 3 000 volts) or the motors (500 to 1 000 volts); while with the former we can "convert," and thus utilize, between the generator and the motor, higher pressures than would be safe at either end. In transmitting currents of electricity over long or short distances, there is a necessary loss in transmission, due to the power required to force the current through the wires. This loss is present in all circuits carrying a current of electricity in greater or less amount, depending upon the volume (amperes) of current (squared), and the quality, size and length of the wire. In cases where large currents are to be carried long distances, the loss is so increased that it requires special consideration. The principle involved in all long-

distance transmission lines is to increase the pressure (voltage) to as high a point as practicable. This is done because the same amount of power can thereby be transmitted, at a correspondingly reduced current requiring a smaller wire, at the same or less loss, since the power is represented by the product of the current and the pressure. Hence, by reducing the current, the energy lost in the line is reduced in proportion to the square of the current. By thus increasing the pressure and reducing the current, the original investment in the transmission line can be kept within reason, and the interest account reduced, as well as the loss of energy in the line. Mr. Davis.

The limit to which this process of reducing the loss in transmission can be carried depends upon our ability to insulate wires carrying high pressures. There is no difficulty in using an alternating current running at a potential of 1 000 to 2 000 volts; for these pressures are common in the outside mains of lighting companies supplying alternating current for incandescent lighting in cities and towns. For long-distance transmission, however, the pressure must be raised to at least 10 000. This is now done successfully in a number of cases. The Niagara Falls transmission line is operated at 11 000 volts, and it is the intention to increase this to 22 000 volts, if it has not already been done; while there are installations on the Pacific Coast where higher potentials are used. How soon higher pressures than those now used will become commercially practicable remains to be demonstrated by the development in the use of present electrical insulating materials, or the discovery of new. In order to obtain high pressures, an intermediate step is required between the generator and the line, known as the "step-up" transformer, and another at the other end of the lines, known as the "step-down" transformer. Generators are built at present to deliver not much more than 2 500 volts, and usually run from 1 000 to 2 000 volts. The "step-up" transformers take this voltage, and raise it, for transmission, to 10 000 or 20 000 volts. At the other end of the line the voltage is reduced to the desired amount, in a similar manner, by a "step-down" transformer and then applied to the distributing lines, or to the rotary transformer, or to both, as the case may require. High insulation is therefore required from the "step-up" transformer along the transmission line to the "step-down" transformer. The required insulation is secured by special construction in the transformers, which are also filled with oil, and by insulators of special design attached to cross-arms and poles adapted for the purpose.

We therefore see:

(1) That the alternating-current motor of to-day cannot be applied directly to operating railroad trains, either on the axles of the cars, or at the head of the train as a locomotive, because of the difficulties in starting under load and in controlling the necessary variations in speed.

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(2) That the alternating current can be applied indirectly to operating railways, because of the facility with which we can transform it to high potentials for long-distance transmission.

Whether it will pay to utilize a "cheap source" such as a waterfall, or concentrate the power into one station on the line of railroad, is, again, the question of balancing the interest account on the added investment for long-distance transmission lines, etc., by reduced operating expenses or increased receipts.

The reduction of first cost to a minimum consistent with work best suited for the purpose is of great importance in the construction of any line. No increase is justifiable, unless it will prove a profitable investment in itself; no decrease, however, is justifiable, if the larger expenditure promises larger profit. Where expenditures can be decreased, without materially lessening immediate traffic or adding to operating expenses, such decrease should be made. The importance of this is seen when we remember that the fixed charges (interest on bonds and rentals) are really a part of the cost of manufacturing transportation—to borrow an expression from an eminent authority; they are, in fact, one of the largest items in the total cost of running a railway system; they increase in a somewhat faster ratio than the cost of the line, and are the same every year, whether business is good or bad; and we therefore emphasize the importance of reducing the first cost to the lowest amount consistent with true economy, and of increasing the traffic to its maximum, so that the fixed charges may be a smaller percentage of the gross receipts. It must be remembered, however, that the gross receipts are often enormously increased by extraordinary expenditures, in which case large investments may be warranted, such as terminals at or near large centers of population, shortening of line for moral effect and saving of time in competition, adopting new methods as an inducement to increased use, etc. An idea of the weight which should be given to this part of the subject will be had by the comparison in Table No. 9.

TABLE No. 9.

	Steam Railroads (U. S.)			Massachusetts Electric Railways.
	Average.	High.	Low.	Average.
Percentage of operating expenses...	65	70	55	69
Percentage of net receipts.....	35	45	30	31
Percentage of fixed charges.....	25	45	14	15
Percentage of profit.....	10	16	4	16

By examining Table No. 9, which is not supposed to show highest and lowest figures, but averages under ordinary conditions, we see the

importance of the item of fixed charges, and the importance, therefore, of reducing the first cost.

The importance of a reduction in first cost, consistent with true economy, is also clearly brought out by:

Fig. 2. Curve 6.—Showing rapid increase in capitalization per mile of track of railways (40% of which is bonded indebtedness, approximately).

Fig. 8. Curve 2-6.—Showing predicted increase in Curve 6 of Fig. 2.

Fig. 3. Curve 4.—Showing greater increase in bonded debt as compared to increase of mileage of railroads.

Fig. 9. Curve 9-4.—Showing predicted increase in Curve 4 of Fig. 3.

Table No. 10. Capitalization of Steam Railroads and Street Railways.—Curve 4. Total Bonded Debt 332% increase 1870–1890, as against 213% increase in mileage of the U. S.; and 240% increase in New York, New Jersey, Pennsylvania, Delaware and Maryland, against 84% increase in mileage, and 437% increase in Massachusetts against 205% increase of mileage. The increase of 500% in bonded debt of street railways of Massachusetts in the same period, against 544% increase of mileage, does not disprove the statement, for this apparent decrease is due to the phenomenal growth of street-railway mileage.

2. Total Expenses.

As first cost is against electric traction we must look to reduced operating expenses or increased receipts as justification for its use. Let us examine the former. Again refer to our tabulated divisions of "total expenses," etc.

We have adopted the title "total expenses" in a literal sense, including all expenses of every kind, so that the difference between "total expenses" and "gross receipts," if on the right side, would give the amount which can be considered the actual net earnings of the line. "Operating expenses," which are generally treated as the usual and ordinary expenses of the system, include "maintenance and renewal of way and works," "train expenses," "station, terminal, taxes and general expenses," and "accidents, loss and damages," but not "fixed charges," although the latter form a great percentage of the actual total expenses, and have become a necessity in our industrial and financial development. The operating expenses include many items which do not vary with the amount of business or the character of the alignment—such as salaries of leading officials; maintenance of way, works and parts of the system, against that deterioration which

TABLE No. 10.—(*Continued*).

							Fig. 2.		Fig. 3.	
	Capitalization of Steam Railroads and Street Railways:									
1	Total miles of track	+	213		84		544		205	
2	Annual increase in mileage	±		11	135		446		4	
3	Total capital stock	+	213		104		215		124	
4	" bonded debt	+	332		240		500		437	
5	" capitalization	+	266		155		248		200	
6	" " per mile of track	+	31		52		20		47	
							Fig. 4.		Fig. 5.	
	Earnings of Steam Railroads and Street Railways:									
1	Gross traffic earnings	+	167		70		406		150	
2	Total interest on bonded debt	+	280		162		715		235	
3	" dividends on capital stock	+	48		16		405		140	
4	Gross traffic earnings per mile of track	+	10		4			**4**	25	
5	Percentage of expenses to gross earnings	+	9		3			**12**		**4**
6	" " interest paid on bonded debt	−	**1**			14		25		15
7	" " dividends " capital stock	−		52		55		10	**6**	
8	Gross earnings per revenue train-mile	−		23				15		10
9	" expenses " " "	−		15				26		23
10	Percentage of passenger earnings to gross earnings	+	8		4				1	
11	" " freight " "	+		6		7				3
12	" " interest and dividends paid on capital stock and bonded debt	−		25					**4**	
							Fig. 6.		Fig. 7.	
	Passenger Traffic of Steam Railroads and Street Railways:									
1	Passenger train-miles	+	750		200		364		204	
2	Total passengers carred	+	766		900		507		308	
3	Passengers carried one mile	+	443		300				202	
4	Gross passenger earnings	+	152		71		100		120	
5	Receipts per passenger per mile carried	−		33		17				30
6	" " " carried	−		35		37		20		28
7	" " " train-mile	−		32		19		15		22
8	" " mile of track	+	30		3			**4**	55	
9	Number of passengers carried per mile of track	+	14		128			**24**	60	
10	Miles traveled per passenger	−		14		20				1
11	Passenger rides per capita per annum	+	300		300		204		100	
12	Receipts per capita per annum	+	58		22		941		50	

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TABLE No. 10.—PERCENTAGE OF INCREASE OR DECREASE OF STEAM RAILROADS AND STREET RAILWAYS FROM 1870 TO 1890.

Curve.	ITEMS.	Characteristic sign of column.	STEAM RAILROADS. UNITED STATES.		STEAM RAILROADS. N. Y., N. J., PA., DEL., MD.		STREET RAILWAYS. MASSACHUSETTS.		STEAM RAILROADS. MASSACHUSETTS.	
			Per cent. Increase.	Per cent. Decrease.	Per cent. Increase.	Per cent. Decrease.	Per cent. Increase.	Per cent. Decrease	Per cent. Increase.	Per cent. Decrease.
	True Valuation of Real and Personal Estate:									
1	Total in the United States	+	117							
2	" " New York, New Jersey and Pennsylvania	+			44					
3	" " Massachusetts	+					33			
4	Per capita in the United States	+	33							
5	" " New York, New Jersey and Pennsylvania	+			0	0				
6	" " Massachusetts	+						16		
7	Percentage of increase in the United States	−		43						
8	" " New York, New Jersey and Pennsylvania	−				85				
9	" " Massachusetts	−						96		
	Population Statistics:		Fig. 1.		Fig. 1.		Fig. 1.		Fig. 1.	
1	Total miles of track	+	213							
2	Annual increase of mileage	+		11						
3	Total population of the United States	+	63							
4	Percentage of increase of the population of the United States	−		28						
5	Density of the population of the United States	+	60							
6	" " " " Massachusetts	+					53			
7	Total urban population of the United States (8 000 and up)	+	127							
8	Percentage of urban to total population of the United States	+	45							
9	Total population of New York, New Jersey and Pennsylvania	+			44					
10	" " Massachusetts	+					53			
11	" " New York City	+			60					
12	" " Brooklyn	+			83					
13	" " Philadelphia	+			55					
14	" " served by N. Y. & P. R. T. R. R., including New York, Philadelphia, Brooklyn, Newark, Jersey City, Camden, Hoboken, Elizabeth, Long Island City, and Richmond County	+			71					

Mr. Davis. is produced only by time, irrespective of the amount of business handled or work done; salaries of agents of all kinds, many of whom have to be employed whether business is large or small, etc. All of these might be called the "fixed operating expenses," and, as they often amount to nearly half of the total operating expenses, it will prove instructive and remunerative to keep railroad accounts so as to show this. To reduce the total expenses to the minimum consistent with true economy is as important as to reduce the first cost to the lowest amount, bearing the same axiom in mind.

We have two cases to consider and compare with steam-railroad operation: First, central steam-power plants located along the proposed line; second, water-power located at some distance from the line. The following applies to both cases, unless otherwise indicated:

"Repairs of earthwork to subgrade" would be unaffected by the system used. Repairs of track, fences, cattle-guards, crossings, etc., renewals of ties, and ballast are substantially independent of the volume of traffic; ties wear independently of the tonnage, for the so-called "cutting" of ties is due primarily to the rotting away under the rail foot, accelerated somewhat, but almost inappreciably, by the decayed material cut away by passing loads. The total cost of track labor is affected only by a very small percentage of the whole by any change in the amount of traffic, provided the standard of maintenance is not increased; rail-renewals, track-watchman, and part of the labor of lining, surfacing, etc., directly applicable to the track itself, are the only items which vary directly with the number of trains. One-half to three-fourths of the wear of steel rails is caused by the locomotive, and is due more to the great concentration of load on the driving wheels, which often nearly approaches the ultimate crushing resistance of the rail, than to the "hammer-blow" of the reciprocating parts. Maintenance of crossings would be unaffected by the system used. For the reasons enumerated above, items 2, 3, 7, 8 and 9 would be unaffected in cost of maintenance, whichever system were used. While some might claim that the renewal of rails would be less frequent on an electric system, it must be remembered that an electric locomotive or a motor car at the head of a train must have, concentrated on the wheels, the same weight for the same speeds and loads, the only advantage being the removal of the hammer-blow of reciprocating parts. Should motors be placed on every axle, there would be a saving. In steam-railroad practice this item amounts to about 4% of the total cost per train-mile (see Table No. 11), so that, should there be a saving of 25%, it would equal about 1% of the total cost per train-mile.

Maintenance of bridges and trestles should be slightly less with the use of electric motors, because of less vibratory effects; maintenance of masonry, buildings and signals would not vary in cost with change

of system. We find under "maintenance and renewal of way and works," that there are no items exclusively applicable to steam-locomotive practice; but there are six which apply to electric traction only. This arises naturally from the fact that these items occur under "first cost," and must be maintained. They are: maintenance of (4) ground-return circuit, (5) overhead-trolley line, or third rail, (6) electric feeder lines, and renewals to (10) poles or insulation of third rail, (11) trolley or third rail and ground return, and (12) feeders. The maintenance and renewals of these constitute an increase in the total expenses of an electric road over those of a road operated by steam. Mr. Davis.

The additional expense of these items, on the basis of all Massachusetts street railways, is shown in Table No. 11. This is, of course, not strictly accurate, as they do not operate under like conditions, but the figures are indicative, so that, with proper consideration of this difference, they can be used:

(*e*) 4.	Repairs of ground-return circuit	0.50	per cent.
(*e*) 5.	" " overhead trolley	1.12	"
(*e*) 6.	" " feeder lines	0.60	"
(*e*) 10.	Renewal of poles for overhead trolley	0.30	"
(*e*) 11.	" " overhead trolley	0 20	"
(*e*) 12.	" " feeders	0.05	"
	Total	2.77	"

The probability is that these percentages are too low for the heavy powers and high speeds of an electric road operating under steam-railroad conditions.

The next division is that of (*b*) "Train Expenses." The relative consumption of coal by steam locomotives and electric motor cars or locomotives, while of importance, is not so large a percentage of the operating expenses as is often imagined. As a rule, the attempt to save in fuel account has had more attention on electric roads than was warranted, and has been made at the sacrifice of other and more important items.

Omitting results obtained from compound locomotives, the records of which are not yet as extensive as those for single expansion, but which should show an average saving of approximately 20%, it has been found that in steam-locomotive practice, of the character we are discussing, the coal consumption per passenger-car-mile (exclusive of engine) averages about 6.5 lbs. for cars weighing 20–25 tons on way-trains, or 30–35 tons on through trains, and that about an average of 1 lb. is added per car-mile for each 6 tons' added weight of car. The average coal consumption of a passenger engine (say 40–45 tons) is about 20 lbs. per locomotive-mile. If we assume six cars to a train, as per our example, we have 3.3 lbs. per car-mile for the engine and 9.8 lbs. per car-mile for the whole train, including engine. This conforms to present average practice of steam railroads.

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TABLE No. 11—COMPARATIVE OPERATING EXPENSES

Ordinary division of items in total expenses.	ALL MASSACHUSETTS STREET RAILWAYS (RAILROAD COMMISSION REPORTS, 1899). Total expenditures. (In nearest $1 000.)	Percentage of total operating expenses.	Operating expenses per car-mile.
(1)	(2)	(3) Per cent.	(4) Cents.
(*a*) *Maintenance and Renewal of Way and Works:*			
1. Repairs of earthwork to sub-grade			
2. Repairs of track	$1 163 000 includes (1), (3), (4), (7), (8), (9), (13).	9.3	1.6
3. Repairs of fences, crossings, etc			
(*e*) 4. Repairs of ground-return circuit			
(*e*) 5. Repairs of electric line (third rail or overhead trolley)	286 000 includes (6), (10), (11), (12), (15).	2.3	0.4
(*e*) 6. Repairs of feeder lines			
7. Renewal of rails			
8. " ties			
9. " ballast			
(*e*) 10. " third-rail supports or poles for overhead trolley			
(*e*) 11. Renewal of third rail or overhead trolley			
(*e*) 12. Renewal of feeders			
13. Repairs of masonry, bridges, trestles, etc			
14. Repairs of buildings	91 000	0.7	0.1
15. " signals			
(*e*) 16. Removal of snow and ice	280 000	2.2	0.4
Totals	$1 820 000	14.5	2.5
(*b*) *Train Expenses:*			
(*s*) 1. Fuel for locomotives			
(*e*) 2. " central power stations	1 268 000 includes (3), (4), (14), (15).	10.2	1.7
3. Water supply			
4. Oil and waste			
(*s*) 5. Repairs of locomotives and tenders			
(*e*) 6. Repairs of electric motors (car equipment)	439 000	3.4	0.6
7. Repairs of passenger cars	727 000	5.8	1.0
(*s*) 8. " freight cars			
(*s*) 9. Use of foreign passenger and freight cars			
(*s*) 10. Locomotive services (wages)			
(*e*) 11. Motor car services (wages similar to 10 and 12 for steam railroads)	4 893 000 includes (12).	39.1	7.0
(*s*) 12. Passenger train services (wages)			
(*s*) 13. Freight train services (wages)			
(*e*) 14. Repairs of central power stations			
(*e*) 15. Central power station services (wages)			
Totals	$7 327 000	58.5	10.3

† Estimated.

OF STEAM AND ELECTRIC ROADS, WITH ITEMS IN DETAIL. Mr. Davis.

Fair average percentage in steam railroad practice.	Percentage of operating expenses on Massachusetts Electric Street Railways by distributing Column 2 according to Column 4.	Probable average percentage of operating expenses all Massachusetts Electric Street Railways estimated from Column 5.	Proportion of percentage of operating expenses of steam railroads, Column 4. Estimated for proposed railroad from Philadelphia to New York (N. Y. & P. R. T. R. R.), to operate in 30 to 35 minutes without intermediate stops.	Probable comparative increase or decrease in cost of operation by a change of motive power from steam locomotives to electric motors.		Operating expenses per car-mile Metropolitan Street Railway Company, New York City, 1899.
				Per cent. Increase.	Per cent. Decrease.	
(5) Per cent.	(6) Per cent.	(7) Per cent.	(8) Per cent.	(9)	(10)	(11) Cents.
1.5	0.6975	0.20	1.5	Unaffected.		.....
8.2	3.8130	4.50‡	5.0	"		0.20
0.3	0.1395	0.02	0.2	"		.....
.....	0.0005	0.50	—	0.50	.....	.....
.....	.....	1.12†	—	1.12	.....	0.29
.....	.....	0.60†	—	0.60	.....	.....
4.0	1.8600	2.50	2.0	.....	1.50	.....
3.0	1.3950	1.24	0.5	Unaffected.		.....
1.0	0.4650	0.30	1.0	"		.....
.....	.....	0.30†	—	0.30	....	.....
.....	.....	0.20†	—	0.20	.....	.....
.....	.....	0.05†	—	0.05	.....	.....
2.0	0.9300	0.04	1.5	.....	0.05	.....
2.0	.....	0.70	2.0	Unaffected.		0.06
1.0	.....	0.03†	1.3	"		.....
.....	.....	2.20	.....	.....	.....	0.13
23.0	.....	14.50	.....	2.77	1.55	0.68
8.0	.....	.....	.....	....	.....	.....
.....	.....	5.40	—	.....	2.60	0.98
0.6	.....	0.30	1.0	.....	0.30	0.14
1.0	.....	0.50	1.0	.....	0.50	0.06†
6.0	.....	.....	.....	.....	.....	.....
.....	.....	3.40	—	0.20	.....	0.475
3.0	.....	5.80	3.0	Unaffected.		1.285
6.0	.....	.....	.....	"		.....
2.0	.....	.....	.....	"		.....
7.4	....	.....	.....	.....	.....	.....
.....	.....	39.10	4.0	Unaffected.		6.46
3.0	.....	.....	.....	"		.....
5.0	.....	.....	.....	"		.....
.....	.....	2.00	—	2.00	.....	0.06
.....	.....	2.00	—	2.00	.....	0.33
42.0	.....	58.50	.....	4.20	3.40	9.79

‡ Paving.

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TABLE No. 11 (*Concluded*).—COMPARATIVE OPERATING EX-

Ordinary division of items in total expenses.	ALL MASSACHUSETTS STREET RAILWAYS (RAILROAD COMMISSION REPORTS, 1899).		
	Total expenditures. (In nearest $1 000.)	Percentage of total operating expenses.	Operating expenses per car-mile.
(1)	(2)	(3)	(4)
(*c*) *Station, Terminal, Taxes and General Expenses:*		Per cent.	Cents.
1. Agents and station services (wages)	.....	.....	.....
2. Station supplies	.....	.....	.....
3. Telegraph	.....	.....	.....
4. Taxes	855 000	6.8	1.1
5. General officers and clerks	625 000 includes (2), (8).	5.0	0.6
6. Legal	119 000	0.9	0.1
7. Insurance	310 000	2.5	0.4
8. Stationery and printing	.....	.....	0.2
9. Agencies and advertising	.....	.....	.....
10. Contingent and miscellaneous, not included elsewhere)	827 000 includes (1), (3), (9).	6.6	1.1
Totals	$2 736 000	21.8	3.5
(*d*) *Accidents, Loss and Damages:*			
(*s*) 1. To freight	.....	.....	.....
2. " passengers	645 000 includes (3), (4).	5.2	0.8
3. " property owned	.....	.....	.....
4. " " not owned	.....	.....	.....
Totals	$645 000	5.2	0.8
(*e*) *Interest Account* (discounts and temporary loans)	523 000	.....	Total 17.1 cents per car-mile.
(*f*) *Fixed Charges:*			
1. Interest on bonded debt	$1 384 000	.....	.....
2. Rentals	1 280 000	.....	.....
Total	$2 664 000	.....	.....
(*g*) *Dividends*	2 076 000	.....	.....

PENSES OF STEAM AND ELECTRIC ROADS, WITH ITEMS IN DETAIL. Mr. Davis.

Fair average percentage in steam railroad practice.	Percentage of operating expenses on Massachusetts Electric Street Railways by distributing Column 2 according to Column 4.	Probable average percentage of operating expenses all Massachusetts Electric Street Railways estimated from Column 5.	Proportion of percentage of operating expenses of steam railroads, Column 4. Estimated for proposed railroad from Philadelphia to New York (N. Y. & P. R. T. R. R.), to operate in 30 to 35 minutes without intermediate stops.	Probable comparative increase or decrease in cost of operation by a change of motive power from steam locomotives to electric motors.		Operating expenses per car-mile Metropolitan Street Railway Company, New York City, 1899.
				Per cent. Increase.	Per cent. Decrease.	
(5) Per cent.	(6) Per cent.	(7) Per cent.	(8) Per cent.	(9)	(10)	(11) Cents.
13.0	3.81	0.09	3.0	Unaffected.		.....
3.0	2.08	0.90	0.5	"		.....
1.0	0.29	.....	1.0	"		.....
3.5	.....	6.80	3.5	"		.....
3.5	2.44	3.80	2.0	"		0.19
0.5	.....	0.90	0.5	"		.....
0.3	.....	2.50	0.3	"		.....
0.7	0.48	0.30	0.2	"		.....
2.0	0.59	0.01	0.5	"		.....
6.5	1.91	6.50	6.5	"		0.76
34.0	.....	21 80	.....	.....	.....	0.95
0.3	.....	.....	.....	Unaffected.		.....
0.4	2.97	5.09	0.4	"		0.53
0.2	1.49	0.10	0.2	"		Included in (2).
0.1	0.74	0.01	0.1	"		Included in (2).
1.0	.....	5.20	.....	.....	.....	0.53
.....	.....	.....	.....	Unaffected.		Total 11.95 cents per car-mile.
.....	.....	.....	.....	Materially affected, estimated from 5 to 15% or more increase.		.....
.....	.....	.....	.....	Unaffected.		.....
.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....

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Table No. 12 shows the average amount of dry steam used per horse-power-hour by various types of stationary engines under different conditions. It is compiled from a large number of actual tests.

TABLE No. 12.—Steam Consumption in Pounds per Horse-Power-Hour.

Type of engine.	Steady Load.		Variable Load 50 to 125 Per Cent.		Extreme Variations—Railway Work, Etc., 0 to 150 Per Cent.	
	Non-con.	Con.	Non-con.	Con.	Non-con.	Con.
High-speed, simple.....	32	28	34	30	36	31
High-speed, compound.....	23	18	25	21	27	22½
Slow-speed, simple..	25	21	28	23	31½	26½
Slow-speed, compound.....	20	15	22½	18	26	23
High-speed, triple expansion.	17½	13	20	16	.....	.....
Slow-speed, triple expansion.	14½	12½	17	15	.....	.....

Tables Nos. 13, 14 and 15 show the coal consumption per car-mile on electric railways, and were compiled from information obtained in 1894–95. Over 600 railways were communicated with, from which 58 have been selected as giving the most accurate results.

Table No. 13 gives the road by index number, the State where located, the car-miles, kind of boilers, engines and coal, the field from which coal was obtained, the long tons of coal used in the given fiscal year, and the pounds of coal consumed per car-mile.

Table No. 14 gives the average, highest and lowest pounds of coal per car-mile, grouping the roads by territorial sub-divisions and kinds of coal, with the number from which the averages have been calculated.

Table No. 15 gives the average, highest and lowest pounds of coal consumed per car-mile for different classes of engine plants, with the number of roads from which the averages have been calculated.

While the ton-mile should properly be the basis, yet the difficulty of determining the number of ton-miles has led to the adoption of the car-mile as the best practical unit of comparison.

Table No. 12 shows a saving of 36% between the high-speed engine plant running non-condensing at 36 lbs. of steam, and the Corliss compound-condensing at 23 lbs. Table No. 15 shows that the high-speed engine plant averages about 15 lbs. of coal per car-mile, while the Corliss compound-condensing averages about 8 lbs.—a saving of 46%, sufficiently close to the 36% given above to indicate that the coal tables give fairly accurate averages. By examining the coal tables we

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are led to the conclusion that the efficiency of each system is approximately the same, for we find, in general, that, where the number of car-miles is high, the consumption of coal per car-mile is low. In the North Central and Western States, where coal is not of as good quality as in the Eastern States, the consumption is higher (assuming, as we safely can, for our purpose, that the economy of various types of boilers is practically the same); with roads using hard coal there is an apparent higher average in consumption of coal, no doubt due to lower car-mileage and fewer cases, and in engine plants, as more economical apparatus is used, the coal consumption decreases. It would seem, therefore, that we are correct in assuming that the total efficiency of each system is approximately the same, though varying in each part that goes to make up the whole. The variation from 25.9 maximum to 5 lbs. minimum shows the average approximate value of this information.

The amount of coal consumed per car-mile is dependent upon so many conditions, often unknown, that this amount, in any given case, cannot be used to predict what the consumption will be on another road; hence the importance of taking a large number of cases upon which to base average results, which may then be considered fairly reliable for the determination of possible results in any new case. It will readily be seen that the amount of coal consumed per car-mile depends upon the tons moved, the height through which each ton is moved, the speed, and the efficiency of the entire system. The first three are almost impossible to obtain on any large number of roads from the ordinary records kept, and can usually be secured only by specially arranged tests. The efficiency of the system depends upon that of each part, such as the boiler plant, engines, condensers and piping, generators, overhead line, ground-return, track, and equipment. High economy in all but one part of the system might cause the pounds of coal per car-mile to be high, and it is therefore impossible, without special test, to determine accurately the cause of low efficiency in any given case. By taking a large number of cases, and eliminating those which are evidently far from the average, we get approximate information indicative of what we may expect.

From Tables Nos. 13, 14 and 15 the average coal consumption per car-mile, under the best conditions of electric street-railway practice, is found to be about 8 lbs. The average weight of these cars, loaded, cannot exceed 8 to 10 tons, so that a six-car train would consume 48 lbs. per train-mile for a maximum weight of 60 tons and average speed certainly not exceeding 10 miles per hour; while with a steam train the consumption would be 58.8 lbs. per train-mile for a maximum weight of 200 tons and average speed of 20 miles per hour. If we compare with compound locomotives, Fig. 10 is of considerable interest. It indicates even a greater saving than 20% which increases as

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TABLE No. 13.

Steam Plant.					Coal.			
Road.	State.	Miles.	Boilers.	Engines.	Kind.	Coal field.	Tons used. (2 240 lbs.)	Pounds used per car-mile
North Atlantic States (As per U. S. Census—1890).								
56 *E*	Maine	116 926	V. T.	V.C.	S.	Virginia	508	9.7
42 *E*	Massachusetts.	196 824	R. T.	C.C.C.	S.	Georges Creek, Md.	939	10.7
65 *E*	"	16 794 961	.....	.....	S.	.....	52 878	7.0
189 *E*	"	917 956	W. T.	H.S.C.C.	S.	Georges Creek, Md.	2 201	5.4
74 *E*	"	873 177	.....	.....	S.	.....	2 262	5.8
451 *E*	"	226 123	R. T.	C.C.	S.	.....	504	5.0
308 *E*	Ohio	1 378 518	.....	.....	S.	Pennsylvania	3 189	5.2
802 *E*	Connecticut	146 000	V. T. / R. T.	H.S.C.	S.	.....	1 000	15.3
398 *E*	New York	819 000	V. T.	V.C.C	S.	Pittsburg	2 176	5.9
78 *E*	"	2 162 134	W. T.	V.C.	H.	.....	8 758	9.1
249 *E*	"	637 290	W. T.	H.S.C.C. / C.C.C.	S.	.....	5 100	17.9
214 *E*	"	199 290	W. T.	.....	S.	.....	1 460	16.4
281 *E*	"	350 000	W. T.	H.S.	S.	Reynolds, Pa	1 200	7.7
181 *E*	"	295 325	R. T.	V.C.C.	S.	Clearfield, Pa	682	5.2
80 *E*	"	7 164 258	W. T.	V.C.C.	S.	Reynoldsville, Pa	23 597	7.4
724 *E*	"	137 970	V. T.	H.S.	H.	.....	1 460	23.7
609 *E*	"	4 558 136	.....	.....	S.	.....	19 043	9.3
842 *E*	"	73 377	R. T.	H.S.C.C.	S.	.....	640	19.5
608 *E*	"	247 482	.....	.....	S.	.....	1 130	10.2
90 *E*	New Jersey	87 923	R. T.	H.S.	S.	Clearfield, Pa	387	9.8
558 *E*	Pennsylvania	149 730	R. T.	H.S.	S.	.....	1 231	18.4
229 *E*	"	600 000	V. T.	V.C.	H.	Hazleton, Pa	3 600	13.4
211 *E*	"	514 989	R. T.	C.	H.	Girardville, Pa	2852	12.4
54 *E*	"	400 648	W. T.	H.S.C.	S.	Dawson, Pa	1 697	9.4
580 *E*	"	501 400	W. T.	H.S.	H.	Schuylkill Co., Pa	2 250	10.0
103 *E*	"	833 000	R. T.	C.C.	H.	Liken's Valley, Pa	3 000	8.1
North Central States (As per U. S. Census—1890).								
648 *E*	Missouri	2 021 938	V. T.	C.	S.	Illinois	16 670	18.5
150 *E*	Ohio	449 680	.....	C.	S.	Jackson Co., Ohio	2 555	12.7
576 *E*	"	331 000	R. T.	H.S.	S.	Wellston, Ohio	1 500	10.1
220 *E*	"	363 040	R. T.	C.	S.	" "	2 300	14.2
222 *E*	Indiana	319 345	V. T.	C.	S.	Illinois and Indiana	3 140	22.0
107 *E*	Illinois	1 514 892	W. T.	C.C.	S.	.....	9 080	13.3
43 *E*	"	530 000	W. T.	H.S.C. / H.S.C.C.	S.	Streator, Ill	4 400	18.6
256 *E*	"	425 090	R. T.	C.	S.	" "	3 560	18.4
250 *E*	Wisconsin	150 000	R. T.	H.S.	S.	Duquoin, Ill	1 500	22.4
594 *E*	"	515 000	W. T.	V.C.C. / H.S.C.C. / C.	S.	Pittsburg, Pa	2 190	9.5
170 *E*	Minnesota	1 133 188	W. T.	C.C.C.	S.	.....	5 510	10.9
148 *E*	Iowa	1 549 060	V.W. T.	H.S.C.	S.	Viclo, Ill	11 400	16.5
257 *E*	Missouri	223 130	R. T.	H.S.	S.	Pittsburg, Kan	2 090	21.0
648 *E*	"	2 021 938	.....	.....	S.	.....	16 670	18.5
825 *E*	"	395 650	R. T.	C.	S.	Crawford Co., Kan	2 150	12.2
743 *E*	"	378 995	R. T.	C.	S.	Pittsburg, Kan	1 980	11.7
53 *E*	Nebraska	100 000	R. T.	H.S.	S.	Kansas	800	17.9
295 *E*	"	1 145 158	V. T. / R. T.	H.S. / H.S.C.	S.	Iowa and Missouri	9 802	19.2

TABLE No. 13—(*Concluded*).

Road.	State.	Steam Plant. Miles.	Boilers.	Engines.	Coal. Kind.	Coal field.	Tons used. (2 240 lbs.)	Pounds used per car-mile.
South Atlantic States (As per U. S. Census—1890).								
811 *E*	D. of Columbia	534 000	V. T.	H.S.	S.	Cumberland, Md....	1 800	7.5
809 *E*	"	365 000	R. T.	C.	S.	"	1 500	9.2
813 *E*	"	528 500	W. T.	H. S. / H. S. C.	S.	W. Virginia.....	2 030	8.6
128 *E*	Georgia.....	98 550	W. T.	H.S.	S.	Alabama	638	14.5
Western States (As per U. S. Census—1890).								
157 *E*	Colorado.....	3 451 136	R. T.	C. / H.S.	H.	Trinidad, Colo.....	25 000	16.2
901 *E*	"	138 375	R. T.	H.S.	S.	Colorado..	1 350	21.8
463 *E*	Utah.....	201 890	R. T.	C.C. / H.S.C.	S.	Rock Spring, Wyo.. / Pleasant Valley, U'h	838	9.3
660 *E*	"	658 416	R. T.	H.S.	S.	Rock Spring, Wyo.. / Pleasant Valley, U'h	3 328	11.3
704 *E*	Washington..	157 832	.....	.....	S.	.....	1 825	25.9
457 *E*	California....	923 694	R. T.	C.C.	S.	.....	2 436	5.9
751 *E*	"	424 529	R. T.	H.S.C.C.	S.	.....	1 250	6.6
Canada.								
796 *E*	Br. Columbia	381 400	R. T.	C.C.C.	S.	British Columbia.....	1 200	7.0
777 *E*	Ontario.....	6 212 045	W. T. / R. T.	C.C.C. / H.S.C.C.	S.	.....	18 535	6.7
364 *E*	Quebec	4 888 486	F.	C.C.C.	S.	Cape Breton, N. B....	16 456	7.5

See note under Table No. 15, for explanation of abbreviations.

the speed rises. Average practice gives about 5.5 lbs. of coal per I. H.-P. per hour in single-cylinder steam locomotives, but the character of the service makes it likely to have enormous variations, in any given run. At times this figure is very much exceeded, and also often falls, under favorable conditions, to 4.5 lbs. The conditions which produce such large fluctuations in steam-locomotive service do not exist in the central power station of an electric road. In the latter case we cannot safely figure on less than about $2\frac{1}{2}$ lbs. of coal per I. H.-P. per hour. This may be reduced in exceptional cases, but will more often be exceeded. Apparently, this is a decided saving in favor of the electric road, but it is only apparent. Let us assume the following efficiencies, which are fair averages: Engine, 90%; dynamos, 90%; line, 80%; and railway motors, 75 per cent. In considering these figures it must be remembered that most of the time the larger part of

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TABLE No. 14.—AVERAGES. NUMBER OF ROADS AND AVERAGE POUNDS PER CAR-MILE.

Locality.	Number.	Pounds Per Car-Mile.		
		Highest.	Lowest.	Average.
ALL ROADS.				
North Atlantic States	26	23.7	5.0	10.65
South " "	4	14.5	7.5	9.95
North Central "	18	22.4	9.5	15.97
Western "	7	25.9	5.9	13.86
Canada	3	7.5	6.7	7.2
Totals and averages	58	....	....	12.46
ROADS USING SOFT COAL.				
North Atlantic States	20	19.5	5.0	10.01
South " "	4	14.5	7.5	9.95
North Central "	18	22.4	9.5	15.97
Western "	6	25.9	5.9	13.46
Canada	3	7.5	6.7	7.2
Totals and averages	51	....	....	10.38
ROADS USING HARD COAL.				
North Atlantic States	6	23.7	8.1	12.78
South " "	..	....	...	.....
North Central "	..	....	...	.....
Western "	1	....	...	16.20
Canada	..	....	...	.....
Totals and averages	7	....	....	13.27

See note under Table No. 15, for explanation of abbreviations.

TABLE No. 15.—AVERAGES (AS PER GROUPING OF ENGINES).

Engine Plant.	No. of Roads.	Pounds Per Car-Mile.		
		Highest.	Lowest.	Average.
C	9	22.0	9.0	14.5
C. C	7	13.4	5.0	9.2
C. C. C	7	10.9	5.2	8.0
H. S	13	23.7	7.5	15.0
H. S. C	3	16.5	9.4	13.7
H. S. C. C	3	19.5	5.4	10.5

NOTE.—The following abbreviations are used in Tables Nos. 13, 14 and 15.

F.	= Flue.	C. C. C.	= Corliss compound-condensing.
V. T.	= Vertical tubular.	H. S.	= High-speed.
R. T.	= Return "	H. S. C.	= " condensing.
W. T.	= Water tube.	H. S. C. C.	= " comp'd-condensing.
V. W. T.	= Vertical water tube.	S.	= Soft coal.
C.	= Corliss.	H.	= Hard "
C. C.	= " condensing.		

the system is running considerably below its rated maximum point of economy; some engineers might place motors at 80%, making the combined efficiency of the entire system about 48.6 per cent. It therefore appears that, for a given average horse-power required at the driving-wheel to move trains, double that amount, or, for safety, even more, must be placed in the central power station, making the coal consumption 5.1 lbs. per I. H.-P. per hour at the motor car, which is but little better than the average of single-expansion steam-locomotive practice, while if compared with compound locomotives (4.4 lbs.) it is higher. It must be remembered, however, that we have been comparing the best electric railway results at low speeds and light weights with average practice in steam locomotives at high speeds and heavy weights; and, while the efficiency of present electric railways is probably less than if they were operated under railroad conditions, it is improbable that this advantage would compensate entirely for the higher speeds and weights which would be moved under such circumstances.

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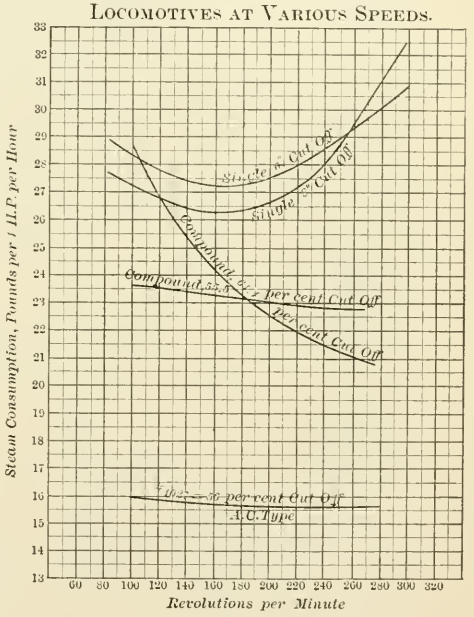
Steam Consumption of Baldwin Locomotives at Various Speeds.
Steam Consumption, Pounds per I.H.P. per Hour
Revolutions per Minute
Single 6" Cut Off
Single 8" Cut Off
Compound 55.6
per cent Cut Off
per cent Cut Off
4 Ubs — 56 per cent Cut Off
A. C. Type

Fig. 10.*

It is interesting to note, as bearing on this subject and as additional indication of the correctness of our position, that a steam road, from which the writer has data, using over 200 000 tons of coal per annum and operating over 42 000 000 car-miles, consumes about 11 lbs. of coal per car-mile in its locomotives. Remembering that the weight moved, in this case, is considerably more than double per car (exclusive of other conditions), and that small detached steam plants are used, running high pressure and single expansion, this is strong corroborative evidence. Turning to Table No. 16 we find that the cost per train-mile on the railroads of the New England States is about 95 cents, or 19 cents per car-mile; this includes freight trains, which, if

* The diagram, Fig. 10, is taken from a paper by Mr. S. M. Vauclain, before the St Louis Railroad Club, November 11th, 1898.

TABLE No. 16—(*Continued*).

—	23	Average distance traveled per passenger (miles)	17	.....	.....	.....	.....	....
+	24	Number of passengers carried one mile (passenger-miles or movement)	1 853 911 708	.....	.....	.....	.....	....
+	25	Number of passengers carried per square mile of settled area	1 892	7 500	3.96	.....	.....	....
+	26	Passenger trips per capita per annum in New England	23	92	3.96	.....	.....	....
+	27	Passenger trips per capita per annum of urban population in towns 8 000 and upward (89 towns)	44	174	3.96	.....	.....	....
+	28	Passenger trips per capita per annum of urban population in towns 2 500 and upward (248 towns)	31	123	3.96	.....	.....	....
+	29	Passenger trips per capita per annum of urban population in towns 1 000 and upward (819 towns)	25	102	3.96	.....	.....	....
+	30	Passenger trips per capita per annum of population served by electric railways, Tables Nos. 3 and 4	.....	.....	.....	43	78	....
+	31	Passenger trips per capita per annum of population served by electric railways, Tables Nos. 3 and 4	.....	.....	.....	74	.....	0.55
+	32	Passenger earnings per capita per annum in New England	7.32	4.64	0.63	.....	.....	....
+	33	Passenger earnings per capita per annum of urban population in towns 8 000 and upward ($)	13.88	8.78	0.63	.....	.....	....
+	34	Passenger earnings per capita per annum of urban population in towns 2 500 and upward ($)	9.78	6.20	0.63	.....	.....	....
+	35	Passenger earnings per capita per annum of urban population in towns 1 000 and upward ($)	8.11	5.14	0.63	.....	.....	....
+	36	Passenger earnings per capita per annum of population served by electric railways, Tables Nos. 3 and 4 ($)	.....	.....	.....	2.21	3.86	....
+	37	Passenger earnings per capita per annum of population served by electric railways, Tables Nos. 3 and 4 ($)	.....	.....	.....	3.72	.....	0.57

(*a*) Reports of 1898 are not at hand at this writing, but for this discussion this is not essential, the relative difference being of greater interest than the actual.

(*b*) This number includes all roads making separate reports to the Railroad Commissioners; many of them are owned or leased to one of the large systems operating in the New England States, but for comparison the above figure is correct.

(*c*) While the average number of cars per train for the New England States is not at hand at this time, the Pennsylvania Railroad, from 1874 to 1884, averaged 5.21 cars per passenger train; since that date the number per train has certainly increased.

(*d*) 5 cars per train.

(*e*) Freight and passengers.

Mr. Davis.

TABLE No. 16.—Comparative Statistics of Steam and Electric Roads.

New England States—Maine, New Hampshire, Vermont, Massachusetts, Rhode Island, Connecticut.

Item number.	Characteristic signs.	Item.	All Steam Railroads. Poor's Manual.	All Street Railways. R. R. Reports and the Writer.	Ratio.	Street Railways in Tables Nos. 3 and 4.		Ratio.
			1897(*a*)	1898.	Street railway returns to steam railroad returns.	Table No. 3.	Table No. 4.	Table No. 3 to Table No. 4.
1	—	Number of roads	145(*b*)	172	1.18	62	34	1.82
2	+	Total passengers carried per annum	109 064 367	432 467 674	3.96	111 839 855	92 965 277	1.23
3	+	Total length of single track operated	7 602	2 393.89	0.32	1 106.45	631.39	1.75
4	+	Percentage of operating expenses to gross earnings	74.77%	67.00%	0.89	65.00	69.00	0.94
5	+	Total passengers carried per mile single track operated	14 347	182 726	12.73	101 079	147 237	0.68
6	+	Total car-miles operated per annum	157 429 890(*c*)(*d*)	83 505 207	0.53	27 682 963	22 457 371	1.23
7	+	Total passenger-train-miles operated per annum	31 485 978	83 505 207 +	2.65	.....	.....	....
8	+	Passenger earnings per mile of single track operated	4 527	9 186	2.03	5 201	7 213	0.72
9	+	Passenger earnings per car-mile operated	21.86(*d*)	23.60	1.08	20.79	20.28	1.02
10	—	Passenger earnings per passenger-train-mile operated (cents)	109.29	23.60+	0.21	.....	.....	....
11	—	Passenger earnings per passenger per trip (cents)	31.55	5.04	0.16	5.14	4.89	1.05
12	+	Total passenger earnings per annum ($)	34 410 597	21 740 618 +	0.63	5 755 217	4 554 674	1.26
13	+	Number of passengers carried per car-mile operated	0.692(*d*)	4.68	6.77	4.01	4.13	0.97
14	—	Number of passengers carried per passenger-train-mile operated	3.46	4.68+	1.35	.....	.....	....
15	—	Percentage of dividends paid	4.96	.....	.....	.....	.....	....
16	—	Percentage of interest paid	4.56	.....	.....	.....	.....	....
17	+	Permanent investment per mile of single track operated ($)	.....	42 717	.....	32 738	42 579	0.76
18	+	Capital stock and debts per mile of single track operated ($)	.....	46 453	.....	32 855	42 031	0.78
19	—	Total operating expenses per car-mile operated (cents)	19.00(*d*)	16.52	0.87	13.20	14.11	0.93
20	—	Total operating expenses per passenger-train-mile operated (cents)	95.00(*e*)	16.52+	0.17	.....	.....	....
21	—	Passenger earnings per passenger per mile	1.86	.....	.....	.....	.....	...
22	—	Number of passenger-miles per passenger-train-mile	58.88	.....	.....	.....	.....	

Mr. Davis. eliminated, would reduce the cost per train-mile and car-mile for passenger trains. Electric railways in the New England States operate at an average of 16.52 cents per car-mile. We also see from Table No. 11 that the average cost of fuel for locomotives is 8% of the total cost per car-mile (passenger and freight), while, for central power stations, it is 5.4 per cent. Combining these figures and allowing for the use of compound locomotives (but making no deduction for freight trains) we have 1.29 cents per car-mile for locomotive fuel and 0.89 cent per car-mile for central power stations. The difference of 31%, apparently in favor of the electric system, in the light of the foregoing data, is due undoubtedly to light weights, low speeds and including freight trains (say 30 cars per train) in the steam-railroad estimate of cost per car-mile, and which would certainly disappear under steam-railroad conditions.

The cost of coal would depend largely upon the amount of power necessary to operate a given road; for the larger the power and more frequent the trains the greater the saving by the use of electric traction. Many central stations, assumed to produce a horse-power at lower cost than by smaller units, really do not, mostly due to the fact that so large a part of such a station is idle many hours of the day; and, although at their maximum point of economy they are far ahead of small units, the question at issue is not "the cost of a horse-power at the station," but "the cost of a horse-power at the rim of the driving wheels." The cost of this last-mentioned horse-power for electric traction is not only made up of station expenses, but also line expenses and electrical equipment expenses, when compared with the cost of the same horse-power produced by a steam locomotive. When these facts are taken into consideration it is invariably found that the controlling factor is the headway of trains; the less the headway, the more chance for electric traction to pay, and *vice versa*, so far as the cost of a horse-power at the rim of the driving wheel is concerned, as affected by the amount of fuel burned. The figures given show this clearly.

The cost of handling coal would be substantially the same with either system, as there would be only one coaling station in either case. The cost of water would undoubtedly be greater in steam-locomotive practice, where it must be supplied at various points along an extended line; but on short suburban roads, to which our discussion is limited, the steam locomotive could make a round trip without requiring water, or would have to be supplied only at each end of the line; and, as the central power station would use much more water on account of larger horse-power and the use of condensing, it is hardly likely that there would be any great difference between the systems. The use of oil and waste on the motor cars should be much less than that which is necessary for the proper care of steam locomotives;

TABLE No. 17. Mr. Davis.

Case 1, Fig. 11.

(*a*) Unlimited amount of water power.

(*b*) Two sources of power, each 100 miles from the line of railway (85 miles long) and dividing the same into ¼, ½ and ¼.

(*c*) Development of these water-powers to cost no more than eleven power stations in Case 2 (this is highly unlikely, but favors Case 1 as against Case 2).

(*d*) Three-phase alternating current of 20 000 volts to be used from source of power to sub-stations, and 1 000 volts direct current from sub-stations to railway motors (this favors Case 1 as against Case 2).

(*e*) Sections 8.5 miles long (44 892 ft.).

(*f*) Sub-sections 11 223 ft. long, 30 in all, each with a sub-station rotary-transformer plant.

(*g*) Three tracks, minimum headway on each, three minutes; trains of five cars each, being 8.5 miles apart.

(*h*) Each train will require not more than 10 000 I. H.-P. at the stations, under economical conditions.

(*i*) Each sub-station an economical rating of 20 000 I. H.-P.

(*j*) In either case, feeder lines along the line of railway equal each other (this also favors Case 1 as against Case 2).

(*k*) All other conditions to be the same as assumed in a previous article by the writer.*

Case 2, Fig. 12.

(*a*) Eleven power stations, one for each section and one at each end of the line, 8.5 miles apart, each supplying current to 3 sub-stations with rotary transformers—sub-stations 2.125 miles apart.

(*b*) Cost of the eleven power stations, each 30 000 I. H.-P., economical rating to equal the cost of developing water powers in Case 1 (favoring Case 1 as against Case 2).

(*c*) Three-phase alternating current of 10 000 volts from power stations to sub-stations, and 1 000 volts direct current from sub-stations to railway motors (this favors Case 1 as against Case 2).

(*d*) All other conditions similar to *e*, *f*, *g*, *h*, *i*, *j* and *k*, of Case 1.

* "The Enormous Possibilities of Rapid Electric Travel." *Engineering Magazine*, 1897.

electric motors can have their wearing parts run in oil boxes, which are a protection against dust and dirt, and automatically lubricate the surfaces subjected to friction, but this cannot be done in the case of steam locomotives. This saving in an electric system is, however, counterbalanced by the use of oil and waste in the central power station, so that it is unlikely that there would be much, if any, difference. The figures in Table No. 11 indicate, however, that there is a saving of about 50% in each of these items as compared with steam railroad results.

Thus far we have considered an electric line with power stations located adjacent to the track and with the usual 500-volt direct-current system. If a cheap source of power (waterfall), located at a distance, with the power transmitted to the line, will reduce operating expenses below what they would be by the assumed method, then this saving can be compared to the results of a steam railroad; but, should the reverse be true, the latter comparison is unnecessary. An example will best bring out the facts, and, while the case is hypothetical, a

Mr. Davis.

careful examination of the figures will satisfy one that the conclusion is correct, whether or not the estimates in themselves are considered accurate.

Fig. 11 shows the layout of a railroad and feeder lines utilizing a cheap source of power (Case 1), while Fig. 12 shows one for which power is generated by steam plants along the line of the road (Case 2).

Assume the conditions shown in Table No. 17.

From Figs. 11 and 12 it will be seen that the track, third rail, direct-current feeders, sub-stations, etc., will be the same in both

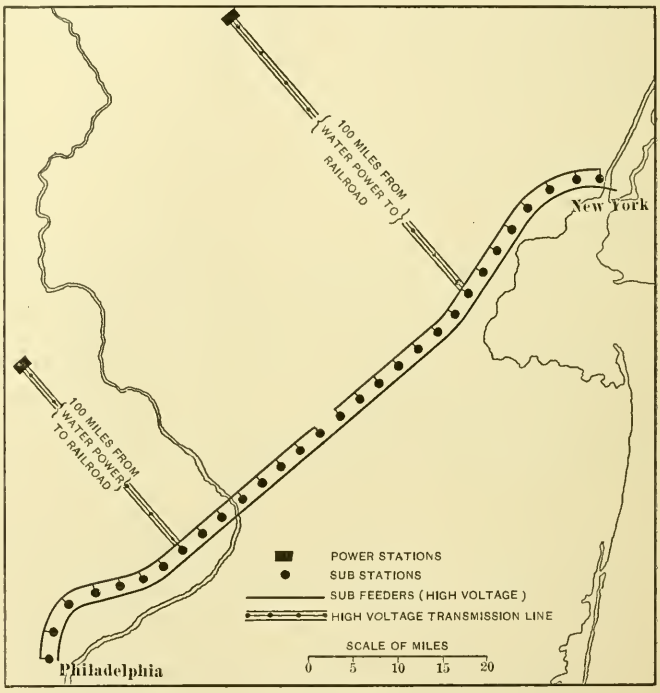

FIG. 11.

cases. The amount of copper in the high-voltage sub-feeders in Fig. 11, which run from the point where the transmission line strikes the road to the sub-station either way, would be greater than that in the high-voltage lines from the power stations to the sub-stations in Fig. 12. The main point to consider, however, is the high-voltage transmission lines themselves (Fig. 11) from the water-powers to the point where they strike the line of the railroad. Under the above conditions, if the interest on the additional investment for these transmission lines from the "cheap source" of power to the railway line in Case 1 exceeds the difference between Case 2 and Case 1 in cost of power delivered to

the sub-station supply feeders, then it is unprofitable to utilize the "cheap source" of power. Mr. Davis.

Our calculations, therefore, can be confined to the cost of the two transmission lines in Case 1, each 100 miles long, from the two sources of water-power to the line of railway, and the difference in the cost of producing power, at the sub-station supply feeders. If the interest on the former exceeds the difference in the latter, then it is unprofitable to use the "cheap source" of power.

In making the calculations, every care has been taken to be fair to the distant source of power, and wherever then is any room for doubt, the "cheap source" has been given the benefit.

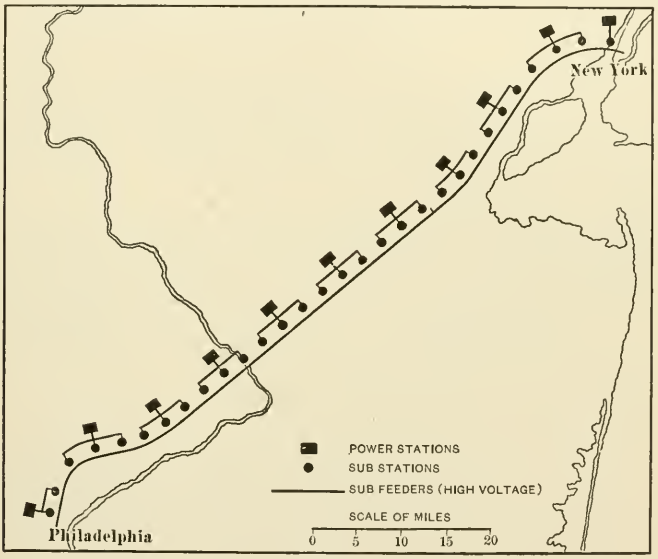

FIG. 12.

The calculation of the pounds of copper necessary is as follows:

$$\text{Circular mils (Diameter of wires squared)} = \frac{\text{Current} \times \text{Distance in feet} \times 20}{\text{Volts lost in line.}}$$

$$\text{Current} = \frac{\text{Watts}}{V} = \frac{\text{Horse-Power} \times 746}{\text{Voltage of transmission.}}$$

$$\text{Pounds of copper} = 0.016 \times \text{circular mils} \times \text{miles.}$$

Therefore,

$$\text{Pounds of copper} = \frac{1\,262\,000 \text{ Horse-Power} \times \text{miles}^2}{(\text{Voltage of transmission}) \times (\text{Loss in volts}).}$$

For 100 miles transmission at 20 000 volts, and a loss of voltage in the line of 20%, or 4 000 volts the formula becomes:

$$\text{Pounds of copper} = 158 \times \text{Horse-Power.}$$

Mr. Davis. This is correct for direct current. For alternating, the inductive loss would increase the amount of copper required to obtain the same total percentage loss, or, if the same amount of copper were used, would increase the total percentage loss. At the same time there would be a saving due to the three-phase wiring system, which would partly counterbalance the loss due to induction. The actual formula would have to be deduced for each special case of wiring, but, on the whole, the above formula, used directly, is sufficient for our purpose.*

As the horse-power to be transmitted equals 412 500 at 20% loss (330 000 H.-P. delivered at the railway line), the total amount of copper necessary is 65 175 000 lbs.

We therefore make the comparison shown in Table No. 18.

TABLE No. 18.

	Case 1.	Case 2.
Allowable loss in pressure from water-powers to railway line	20%	.....
Total horse-power to be delivered to railway line at supply feeders of sub-stations	330 000	330 000
Total horse-power at source of power	412 500	330 000
Number of power stations	2	11
Total horse-power at each power station	206 250	30 000
Total distance power is transmitted from each station (miles)	100	0
Pressure at source of power (volts)	20 000	10 000
" " railway line (volts)	16 000	10 000
" lost in line (volts)	4 000	0
Number of sub-stations supplied from each source	15	3
Average length of sub-feeders (miles)	10.625	4.25
Number of sub-stations in each group of sub-feeders	7.5	3
Pounds of copper in transmission line	65 175 000	0

To the total cost of transmission line, amounting to $25 000 000 (in round numbers), should be added the additional cost of developing the two water-powers to 412 500 H.-P. as against 330 000 H.-P. in the eleven steam-power stations along the railway line. There should also be added the additional cost of feeder lines from the two railway terminals of the transmission lines to the sub-stations, as against the cost of feeders from the steam-power stations to the sub-stations. While these two items would undoubtedly amount to several million dollars in the case under discussion, we have omitted them, owing to the difficulty of estimating the former without accurate surveys of special localities to be developed; their omission favors Case 1. So far, we have an excess in first cost of $25 000 000 against Case 1 as compared with Case 2.

---

*The correct formula is: Circular mils $= \frac{D \times W}{P \times E^2} \times K$;

where D = distance one way in feet,
W = total watts delivered to consumer,
P = percentage of loss in line of W,
K = 1 690,
and E = potential between conductors at consumers' end of line.

Table No. 19 is an estimate of the cost of our proposed transmission line. Mr. Davis.

TABLE No. 19.

Right-of-way—two strips 100 ft. wide and 100 miles long = about 2 500 acres at $1 000 average	$2 500 000
Engineering, 2% on $15 000 000	300 000
Clearing 600 acres at $100	60 000
Bridges, viaducts, or submarine cables, say (an approximate estimate)	1 500 000
Fencing	200 000
Crossings, say 120 at $1 000	120 000
Signals for guarding and repairing 200 miles at $1 000	200 000
Telegraph, 200 miles at $1 000	200 000
Narrow-gauge repair track and equipment, 200 miles at $5 000	1 000 000
Feeders, 65 175 000 lbs. at 15 cents in place	9 780 000
Pole line, 11 000 poles set in concrete at $100 each	1 100 000
Discount on $25 000 000, 3% 100-year bonds	1 250 000
Interest on bonds to opening of road, say 10 years of construction, averaging $2 500 000 per annum at 3%	4 125 000
Taxes to opening of road, assuming ¼ assessment and 2% rate	1 375 000
Office expenses, salaries, etc., to opening of road, 2% on $15 000 000	300 000
Contingent, etc., not itemized, to opening of road, 5.2%	1 300 000
Total cost	$25 310 000

Table No. 20 is a comparison of the running expenses under the two conditions, based upon the calculations in "The Enormous Possibilities of Rapid Electric Travel," already referred to, the items being given in percentages of the total cost per car-mile run, and being reasonable averages (Table No. 11, Column 7).

TABLE No. 20.*

	Case 1.	Case 2.
Repairs of N. G. track on transmission line, right-of-way	1.0%	.....
Repairs of fences, crossings, etc., on transmission line, right-of-way	0.1%	.....
Repairs of feeder lines on transmission line, right-of-way	6.0%	.....
Renewals of poles and insulation on transmission line, right-of-way	1.5%	.....
Renewals of feeder on transmission line, right-of-way	1.0%	.....
Repairs of bridges and trestles on transmission line, right-of-way	0.3%	.....
Repairs of buildings on transmission line, right-of-way	0.1%	.....
Repairs of signals and telegraph on transmission line, right-of-way	0.5%	.....
Fuel for power stations	.....	8.0%
Water supply	.....	1.0%
Oil and waste	0.5%	1.0%
Repairs of power stations	2.0%	4.0%
Power station wages	2.0%	4.0%
Average horse-power developed at source of power	412 500	330 000
Total car-miles per annum	43 435 000	43 435 000
Total cost per car-mile	.....	15 cts.
Add for excess power developed by water-powers (25%) say	3.0%	.....

* All other items being the same, the reader is referred to the other articles mentioned in this discussion.

Mr. Davis. From Table No. 20, it is seen that the items added to the cost per car-mile by the use of water-powers located 100 miles away about balance the greater cost of fuel, water, oil and waste, repairs to power stations, and power-station wages, where the stations are located close to the tracks and operated by steam power (each amounts to 18 per cent.). Hence the total expenses would be practically the same for each case. As the transmission lines cost $25 000 000, the interest thereon is the total increase in the expense of utilizing the water-powers instead of steam-power plants. This interest may be figured at 3% ($750 000), 4% ($1 000 000), 5% ($1 250 000), 6% ($1 500 000), or even 10% ($2 500 000), as the case may require, or good judgment dictate. The use, therefore, of these supposed water-powers would decrease the earning capacity, instead of increasing it.

Let us, for the moment, assume that our calculations of the operating expenses with the use of water-powers is inaccurate, that in many of the items there is a greater saving, and that those having no counterpart in the other column can be reduced materially.

TABLE No. 21.

Interest.	Car-miles.	Total Interest.	Saving in Cents per Car-mile which must take place to allow the investment.
10%	43 435 000	$2 500 000	5.81
6%	"	1 500 000	3.68
5%	"	1 250 000	2.90
4%	"	1 000 000	2.32
3%	"	750 000	1.74

But 18% (total of percentage items in former table) of 15 cents (assumed cost per car-mile) equals 2.7 cents; so that, at the five interest rates, we can afford to have our figures wrong by the percentage shown in Table No. 22.

TABLE No. 22.

With interest at 10% — 266% of 18% = (38% of total)
" " " 6% — 133% " " = (24% " ")
" " " 5% — 105% " " = (19% " ")
" " " 4% — 83% " " = (15% " ")
" " " 3% — 64% " " = (11% " ")

It is unlikely that we have made any such error as would be necessary to show that, even with money worth only 3%, it would pay to utilize the water-powers in the case we have discussed. Furthermore, we have favored Case 1 in every way, and more especially by comparing a 20 000-volt system with one of 10 000 volts.

Had we used higher voltages on our transmission line, the copper would have cost:

At 50 000 volts 63 × H.-P. = 10 310 000 lbs. $1 555 000.
100 000 " 25 × H.-P. = 2 600 000 " 390 000.

By using higher voltages some items of the first cost would be reduced slightly, but, as these constitute more than 60% of the total cost, whereas copper constitutes less than 40%, a reduction in the latter increases the percentage of the former, although reducing the actual amount. Using 100 000 volts, the first cost would not be less than $10 000 000; the interest on this at 3% equals $300 000, and the corresponding saving per car-mile which must take place to allow this investment equals 0.7 cent; therefore 25% of 18% (4.6% of total) can be allowed, even in this case, for errors in our calculation. Mr. Davis.

The case we have discussed shows conclusively that it would not pay to utilize the water-powers. But, it may be said, this covers only one problem. While this is true, nevertheless this problem is one chosen to enable us to arrive at a general conclusion; for we are dealing with a very large amount of power at a comparatively short distance. The larger the power to be used, the more likely it is that a cheap source will prove profitable; the greater the distance over which the power must be transmitted, the less likely it is that a cheap source will prove profitable.

These calculations have been given to bring out clearly how unlikely it is that, even by the use of alternating long-distance transmission, a trunk line, like the Pennsylvania Railroad, for example, will convert its entire system to electric traction. This system, east of Pittsburg, has 2 747.35 miles of railroad, 8 259.36 miles of single track, 1 803 locomotives, 1 765 passenger, baggage and express cars, and 90 527 goods cars. If locomotives were converted at an average horse-power of 300 each, this would make an aggreate of 216 360 H.-P. to be transmitted (allowing 20% in shops and 50% of the total horse-power of the locomotives to be transmitted), while it would have to be taken over many times the distance given in the example. Apply the formula for pounds of copper in transmission line and we see how enormously the weight is increased as the distance increases in the case of the Pennsylvania Railroad. The cost is astonishing when one remembers the claims of an enthusiast who stated: "Where water-power is always available within a few hundred miles" or "for desert railways, where water cannot be obtained," "electric traction is eminently suitable."

Repairs of passenger locomotives and tenders on steam roads average from 4 to 6 cents per mile, with about 1.5 cents to be added for repairs to tools, shops, machinery, etc., properly chargeable to "motive power repairs." Statistics of electric roads are very imperfect and unreliable on matters of this kind, not only because, in the past, such roads have usually been operated by those familiar with horse traction, and having little mechanical knowledge of or inclination toward the precise methods of steam-railroad management and operation, but because of the constant change of apparatus long before the old had given the life which could reasonably be expected of it as a piece of mechanism.

Mr. Davis. Even if we had such data as we might wish from existing electric roads, they would be only indications of what might be expected on a system similar to the one under discussion. From Table No. 11 we find that locomotive and passenger car repairs amount to 9%, while motors, trucks and cars on the electric railroads of Massachusetts cost 9.2%, which again favors the steam railroads; for to this must be added 2% for power-plant repairs, making a total of 11.2 per cent.

When we remember the small horse-power involved, and the light weights and low speeds, it is hardly likely, with the large increase in these necessary under steam-railroad conditions, that the cost of repairs and maintenance of the motive power of an electric road will be any less than that of a steam road. It is to be regretted that more reliable and extensive data are not obtainable. Repairs and maintenance of passenger cars would be the same in either system. The item "use of foreign passenger cars" is the rental paid for the use of cars of other roads while retained, and of course would be unaffected by the use of any particular system. While the operation of a motor car involves far less labor than that of a steam locomotive and its tender, yet the danger that the operator might be incapacitated makes it necessary to have two men at the head of the train, although one man would easily be able to do all the required work. As it takes two men to run the ordinary passenger locomotive, there would be no advantage of one system over the other. It would be a mistake to save wages by utilizing a lower degree of intelligence or skill for motormen and their assistants than is now used for engineers and firemen on steam railways; the tendency to do this has cost electric street-railway owners many thousands of dollars—more, in fact, than is usually realized. Passenger-train service would not be affected by the use of one system or the other. Central-power-station service is an additional charge against the electric road. Were the operative method changed, train services (wages) would be greatly increased, as shown in Table No. 11.

We now come to (*c*) "Station, Terminal, Taxes and General Expenses," divided into ten subdivisions, none of which is affected by the character of road-bed, rolling stock or mechanical apparatus of any kind, but is dependent upon the character of the business handled, the mode of operation and the volume of traffic; therefore these items are unaffected by the use of any particular motive power. The same statements apply to the division (*d*) "Accident, Loss and Damage," and to (*e*) "Interest Account."

"Fixed charges" includes interest on bonds, and rentals; the actual rate of interest on bonds is not affected by the system used, except in ways beyond the scope of this discussion. We have shown that the first cost of an electric system is considerably greater than that of a steam-locomotive road when operating under the same conditions, and

only in one case will it be less, namely, when the units are small, light and very frequent, when the cost of locomotives would more than equal the natural increases caused by electricity. Mr. Davis.

Therefore the "fixed charges" of an electric system, under steam-railroad conditions will be much higher, and this may vary from 5 to 15% or more as an increase of expenses against the electric road, assuming, of course, that the cost in each case is covered by bonds, which is now usual.

This is no inconsiderable item, as the average fixed charges of our present steam roads are not less than 20% of the gross receipts, and, if this is increased largely by the use of electricity, it is more of a burden than can be overcome by inconsiderable savings in items of operating expenses (if such savings are even possible), such as slightly lower coal consumption, motor car repairs and maintenance, bridge repairs and maintenance, track repairs and maintenance, etc., which may be claimed in favor of electricity by some ardent advocates, irrespective of the intrinsic merits of any given problem.

To recapitulate: We see, by Table No. 11, that the additional charges against electric traction, omitting interest on increased fixed charges, amounts to 6.97%, while the savings, allowing the maximum in each case and all doubtful items, is 4.95%; or 2.02% against the electric system, with the largest of all items not considered.

From what precedes we have shown that the following would be additional charges against the electric road over a steam railroad, and that they would not be inconsiderable, especially the interest charge:

1. Interest on increased cost.
2. Renewal and repairs of transmission line.
3. Repairs of motors and central power plants.
4. Central-power-station services.

And the following the possible, but not necessarily probable, savings:

1. Renewal of rails.
2. Repairs of bridges, trestles, etc.
3. Fuel for central-power plants (unlikely).
4. Water supply (unlikely).
5. Oil and waste (unlikely).

These are items of sufficient importance to necessitate a very careful study of each individual problem before deciding upon a change, or even an original installation.

But some may say that the average of 17 cents per car-mile is high for electric roads, and so it is for a first-class well-managed road, but so is 19 cents for a similar steam road. The Metropolitan Street Railway of New York makes the statement that they operated at 11.95

Mr. Davis. cents per car-mile, while in an article in *Cassier's Magazine* of August, 1899, the statement is made that the Chicago South Side Elevated operated, in the month of December, 1898, at 7.5 cents per car-mile. Both these figures must be taken with a great deal of caution, and for the same reason, namely, the period during which both systems have been in operation has been short, and neither yet knows what its true operating expenses are. Table No. 11 (last column) indicates this. In this table it is seen that the "train expenses" of the Metropolitan Street Railway, most items of which are unaffected by time, are approximately equal to the average of all Massachusetts roads, which have been running several years, while "maintenance and renewal of way and works" are about 25% of the Massachusetts roads. "General expenses" are also about 25%, and accidents about 70 per cent. General expenses were distributed between cable, horse and electric systems; but, assume that these figures are correct, and that an electric road can be operated at 7.5 cents per car-mile, even then they are not approaching the best steam railroad results. The Pennsylvania Railroad Division of that system operated, in 1897, about 5 cars per passenger train and 30 cars per freight train (both figures are probably underestimated); a total train mileage of about 50 000 000 was made, divided into 16 000 000 passenger-train-miles and 34 000 000 freight-train-miles. The cost per train-mile was about 75 cents for the total mileage. Taking 75 cents as the actual cost per train-mile, for either passenger or freight, we would have 15 cents per car-mile as operating expenses per passenger cars and 2.5 cents per car-mile for freight. This method of estimating is obviously wrong, for a freight-train-mile must cost more than a passenger-train-mile, but the former would have to rise above the average to $2.25 per freight-train-mile before the cost per car-mile would increase beyond 7.5 cents. Again, the average weight of an electric car may be taken at 20 000 lbs. (10 tons), so that at 7.5 cents per car-mile the operating expenses would be 0.75 cent per ton-mile, while the Pennsylvania Railroad Division operated at 0.311 cent per ton-mile.

In making a comparison in the use of steam locomotives and electric motors as motive powers, it is seen that only where the units are many, light and frequent, and operated over comparatively short distances, can the use of electricity result in lower first cost or operating expenses; but this involves a change in the method of operation for it to prove desirable. Where units are few, heavy and infrequent, and operated over long distances, the use of steam locomotives will result in lower first cost and operating expenses, and prove more desirable. Though this general statement may be modified by special conditions, such as very high speeds at frequent intervals, it will generally govern in determining changes, or the choice of one or the other, in the case of newly projected roads.

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Without trying to define the exact limit beyond which electricity would be injudicious, we can state that, except for suburban traffic, and interurban traffic between towns a short distance apart, electric traction is to-day higher in first cost and operating expenses; and even when used under such conditions, it may still be so, and success will only come with a corresponding change in operative methods.

The fact that usually the first cost and the total expenses will be greater with electricity than with steam might lead us to discard the thought of using the former under any circumstances, were it not for the question: How will our gross receipts be affected by the use of one or the other system? If by using electric traction we can increase sufficiently the gross receipts per car-mile or train-mile, and per mile of road, we can afford to pay for the additional first cost and greater total expenses. This is the vital question and the real one at issue, although usually not so considered; but, in general, the inducement to adopt electricity instead of steam, or to discard the latter, will be the increased travel which undoubtedly comes, in a greater or less degree, to roads using electric motors.

Freight and miscellaneous receipts cannot be affected by any change of motive power, so our discussion will be confined to the effect on passenger receipts.

3. Gross Receipts.

By examining statistics of steam railroads we find that the volume of travel is influenced by such features as the following: Convenient location of stations with respect to centers of population; length of line; proximity of terminals to the centers of population; number of trains; (the four points just mentioned have an overwhelming effect on short-haul traffic); good road-bed and track neatly kept; handsome commodious stations, with first-class appointments and service; comfortable and luxurious cars; block signals; cleanliness, etc. Profit is proportional to success in obtaining that travel which enables each train to run full, for the cost of a train-mile is not affected by the number of seats taken; and it is this travel which is obtained by a line giving weight to its "location" and "appointments." In freight competition between railroads the rates are ultimately dependent upon the cost from "consignor to consignee," not from station to station, and the railroad eventually pays the additional cost of lighterage, switching, cartage, etc., and the profit thereon, whether by actual difference in rates or loss of business.

The same is true in passenger traffic, as will be seen by free omnibuses, reduction in rates, etc. If you can take the passenger up at his own door and set him down at his place of destination, you have not only suited his convenience (and thus, as we shall see, induced him to travel oftener), but have secured those receipts which otherwise

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TABLE No. 23.—POPULATION

STATES.	Population (United States Census, 1890).	Density (population per square mile).	Total area (square miles).	Area of settlement (square miles).	URBAN POPULATION (CALCULATED FROM U. S. CENSUS 1890.)			
					Towns 8 000 and upward.		Towns 2 500 to 8 000.	
					No. of towns.	Total population.	No. of towns.	Total population.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Maine	661 086	22.11	29 895	25 729	8	130 346	39	152 677
Connecticut	746 258	154.03	4 845	4 845	17	385 287	49	204 923
Massachusetts	2 238 943	278.48	8 040	8 040	47	1 564 931	103	445 018
Rhode Island	345 506	318.44	1 085	1 085	10	272 571	13	55 439
New Hampshire	376 530	41.81	9 005	8 328	5	103 058	22	85 137
Vermont	332 422	36.39	9 135	9 135	2	26 350	22	90 713
Totals and averages	4 700 745	79.79	62 005	57 662	89	2 482 543	248	1 033 907

would have gone to omnibuses, hackmen, and street-car lines. This the steam railroads have failed to do, and it is clear that they cannot altogether do away with these feeders natural to their peculiar modes and conditions of traffic ; but there is little room to doubt that in many cases steam railroads can modify their present methods, for suburban traffic out of large centers of population and for interurban passenger traffic on branch lines between centers of population, by the use of electricity, paying the additional first cost and greater total expenses out of the probable (we might say certain) enormous increase in their gross receipts. This, the writer believes, can be accomplished only by a radical change in the present methods of operation, making them approach, on parts of the system, the present "leave-at-your-door" plan of our street railways, while keeping, on the rest of the system, the present methods of steam railroads with possibly some minor modifications. To accomplish this to advantage, the use of electricity will probably prove advisable, although in some instances a combination of electricity with steam might give the best results. The great increase in gross receipts and in net revenue may, and very often will, decide the question beyond any possible doubt in favor of electricity, always remembering that the adoption of electric traction means a corresponding adaptation of operative methods, and not the mere application of a special form of motive power.

Let us consider some of the figures bearing on the problem.

The Pennsylvania Railroad originally spent about $5 000 000 for its Broad Street terminal; the great St. Louis Bridge and Union Station

STATISTICS (UNITED STATES CENSUS). Mr. Davis.

URBAN POPULATION (CALCULATED FROM U. S. CENSUS, 1890).

Towns 1 000 to 2 500.		Total in Towns 1 000 and upward.		Balance of population in States not in towns, 1 000 and upward.	Tables Nos. 3 and 28.		Tables Nos. 4 and 29 (omitting *C* 13).		Tables Nos. 3 and 4 (omitting *C* 13).	
No. of towns.	Total population.	No. of towns.	Total population.*		No. of towns.	Population served.	No. of towns.	Population served.	No. of towns.	Population served.
(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
143	209 864	190	492 887	168 199	25	175 676	9	90 219	32	210 887
56	89 894	122	680 104	66 154	52	326 252	57	401 858	77	474 420
107	172 984	257	2 182 933	56 010	144	1 850 837	32	581 668	152	1 961 280
9	14 112	32	342 122	3 384	.....	.....	....	.....	.....	.....
74	105 018	101	293 213	83 317	.....	.....	7	106 673	7	106 673
93	135 831	117	252 894	79 528	.....	.....	....	.....	.....	.....
482	727 703	819	4 244 153	456 592	221	2 352 765	85	1 180 418	268	2 753 260

cost many more millions; while the Market Street terminal of the Philadelphia and Reading Railroad, the Union Station of the Boston and Maine Railroad systems, the Southern Union Station of the New York, New Haven and Hartford Railroad in Boston, and the Grand Central Station in New York are proofs of the millions of dollars spent by railroad companies to locate their terminals at or near the centers of population. These expenditures were incurred almost entirely for the sake of increasing the suburban passenger traffic, although giving a decided stimulus to through and competitive business. In the writer's opinion, our steam railroad companies can afford to double (to use a broad and inaccurate expression) these investments to accomplish what has been suggested above, reaping a handsome return from the increased gross receipts from the passenger traffic thus obtained and encouraged.

Taking the New England States as a basis for comparison—for there we shall find the shortest hauls, the most dense population, the greatest number of trains, and the most accurate statistics—we have from Poor's Manual, Railroad Commissioner's Reports and special calculations by the writer the various tables and figures hereafter discussed.

Effect of Increased Population.—Fig. 1 shows clearly the enormous growth of population and railroad mileage in the United States since 1830, when steam railroads were just coming into existence. Street railways started at about the same time, the first street car operating on the New York and Harlem Railroad tracks, Fourth Avenue, New

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TABLE No. 24.—Growth of Street Railways in 10 Years, 1888 to 1898 (*a*).

Motive Power.	Number of Railways.		Miles of Track.				Number of Cars.				Average Length of Railway (Miles of Track).	
					Percentage to total.				Percentage to total.			
	1888.	1898.	1888.	1898.	1888.	1898.	1888.	1898.	1888.	1898.	1888.	1898.
Electric.....	21	914	86	14 782	1.4	90.2	172	41 402	0.6	87.8	4.09	16.17
Cable.....	18	20	217	486	3.6	3.0	2 777	2 920	11.1	6.2	12.06	24.30
Horse.....	566	112	5 474	683	91.4	4.2	21 736	2 860	86.6	6.0	9.67	6.09
Steam.....	35	27	216	421	3.6	2.6	(*b*)423	(*b*)404	1.7	0.8	6.17	16.59
Totals*..	640	1 073	5 993	16 372	.....	.....	25 108	47 586	.....	.....	.....	.....

(*a*) From an article by Mr. William J. Clarke, entitled "Electric Railways in America from a Business Standpoint."—*Cassier's Magazine*, August, 1899.

(*b*) Includes locomotives.

This table is evidently approximate, and was undoubtedly not intended to be exact.

York City, in November, 1832. It would be curious if in the near future (70 years after) some steam railroad should again change its mode of operation so as to approach the methods of street railways. Curves Nos. 7 and 8 show the growth of urban population (according to United States Census those communities which have 8 000 or more inhabitants), while Table No. 23 shows the urban population in Massachusetts down to towns of 1 000 inhabitants.

It is unfortunate that the old horse railways have left so few records, but Fig. 2, showing the increase of mileage in Massachusetts from 1870, gives an indication, while Table No. 24 shows the growth of street railways in ten years, 1888 to 1898. The mileage of street railways (16 000) when compared with that of steam railroads (182 000) shows that the former are confined to thickly populated areas, either within their limits or connecting two or more such areas. Their methods of operation—small, frequent units—adapted itself to such distribution of inhabitants and not to the longer hauls at infrequent intervals shown by the mileage of steam railroads.

The growth of population and industries of all kinds (including transportation) react on each other, favorably or otherwise, depending upon conditions of prosperity. A system like the Pennsylvania Railroad could not have existed in 1830, even though it could have been operated at as low a cost as to-day. It had no suburban short-haul business in its early days.

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The concentration of population into "urban" districts has come through the increased transit facilities of such communities. The horse-roads first enabled a growth which made it necessary for the steam railroads to increase their local facilities, which again reacted on the population to increase it, and then came the cable road to reduce the time spent in getting in and out from the residential sections to the business centers; this, in turn, increased the radius of urban population, and again the steam railroads reached out farther from the center of population; all of which encouraged concentration of inhabitants, and the electric railway once more caused a similar cycle of events.

There is no reason to suppose that these conditions have ceased, and that this action and reaction have stopped. It is this enormous development of the country which has allowed the constant increase of investment on transportation systems, and this in turn has increased the carriage of both passengers and freight. In the growth of transit facilities and methods the "radius of action" of the roads has increased constantly, and as it increased so did the urban population and *vice versa*; but when our cities grow beyond the present "radius of action," our transit systems must take another step. Some communities have already arrived there, and the solution will undoubtedly be the equipment of our steam railroads, running out of and between those near together, with electric traction, thereby enabling them to change operative methods to approach present street railway practice, thus increasing gross receipts enough to not only offset interest on increased first cost, but reduce operating expenses per car-mile; for the increase in present investments and traffic have also tended to reduce the cost of transportation per head.

Effect of Increased Capitalization.—Fig. 2 also shows the enormous increase in mileage and investment of street railways in Massachusetts, where the density of population is high and has increased more rapidly than in any other part of the United States, as shown by Curves Nos. 5 and 6, Fig. 1. It is because of this increase, which is still going on, that constant increased investments are warranted on established transit systems, and which will induce our steam railroads to handle increased traffic by small, frequent units in urban, suburban and interurban areas, operated by electric motors, and thus also stimulate and again increase a traffic which will only grow so much and then wait to be encouraged.

Curve 6, Fig. 2, shows the tendency toward increased investment per mile; the drop from 1894 to 1896 indicates a greater increase in suburban than urban mileage, costing less per mile and thus reducing the average. Fig. 3 shows how comparatively small was the percentage of increase of steam railroad mileage, but a very distinct increase is shown in investment per mile (Fig. 3 gives totals of roads in Massachusetts, but includes mileage, etc., outside the State).

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Effect of Increased Earnings.—Figs. 4 and 5 show the earnings of street and steam roads in Massachusetts. Attention is also called to Table No. 16, giving comparative statistics of steam and electric roads, and also to Table No. 10, giving the percentage of increase or decrease in all items (curves) discussed and found in the various figures, between the years 1870 and 1890. We again see the constant increase in gross receipts, especially of the street railways (400% as against 150% for freight and passengers of steam railroads, being 120% for passengers only, Fig. 7, Curve 4); and also the increase in interest and dividends on increased capitalization, some of which is not chargeable to increased mileage (also much larger for the electrics than for the steam).

The gross traffic earnings per mile increased 25% on the steam and decreased 4% on the street roads; the latter is undoubtedly due to the relatively large increase in mileage; this curve, however, has a "characteristic + sign" which will be shown in future growth in the years to come. Curves 8 and 9 in both figures show the characteristic constant reduction in receipts per car-mile or train-mile, due to increased mileage giving better facilities and thereby increasing the traffic at a slightly lower rate, both as to numbers and fares; also reduction in operating expenses per car-mile as the same increases.

Effect of Increased Traffic.—Figs. 6 and 7 (see Table No. 10 also) again bring out the same characteristic comparisons as the previous figure, and, in addition, the following: Curve 10, Fig. 7, indicates that passengers are hauled a shorter distance, on the average, every year. Fig. 13 brings this out somewhat more clearly. In the Eastern States, which are more densely populated, the average distance is approximately constant, but with a tendency to fall. In other words, the proportion of short-haul passengers to total passengers is rising as the population increases and the rides per inhabitant per annum increases.

As the long-distance telephone system develops, its use will grow to such a point that it will no doubt have a decided effect on long-distance passenger traffic, if it has not already. This will increase the proportion of short hauls as compared with long hauls, and the passenger business of our steam railroads (or their partial development into electric railways) in the future, will be more and more largely that of suburban and interurban localities. The increase in total passengers carried was 308% on the steam railroads and 507% on the street railways (see Table No. 16 for totals), while passenger-train-miles increased 204% and 364% (car-miles) respectively.

The receipts per passenger are constantly decreasing as the number of passengers increases, as shown by Curve 6 on each figure. The larger proportional increase in mileage of street railways has reversed the characteristic sign of Curve 4, Fig. 4 (receipts per mile of track), and Curve 9, Fig. 6 (number of passengers carried per mile of track),

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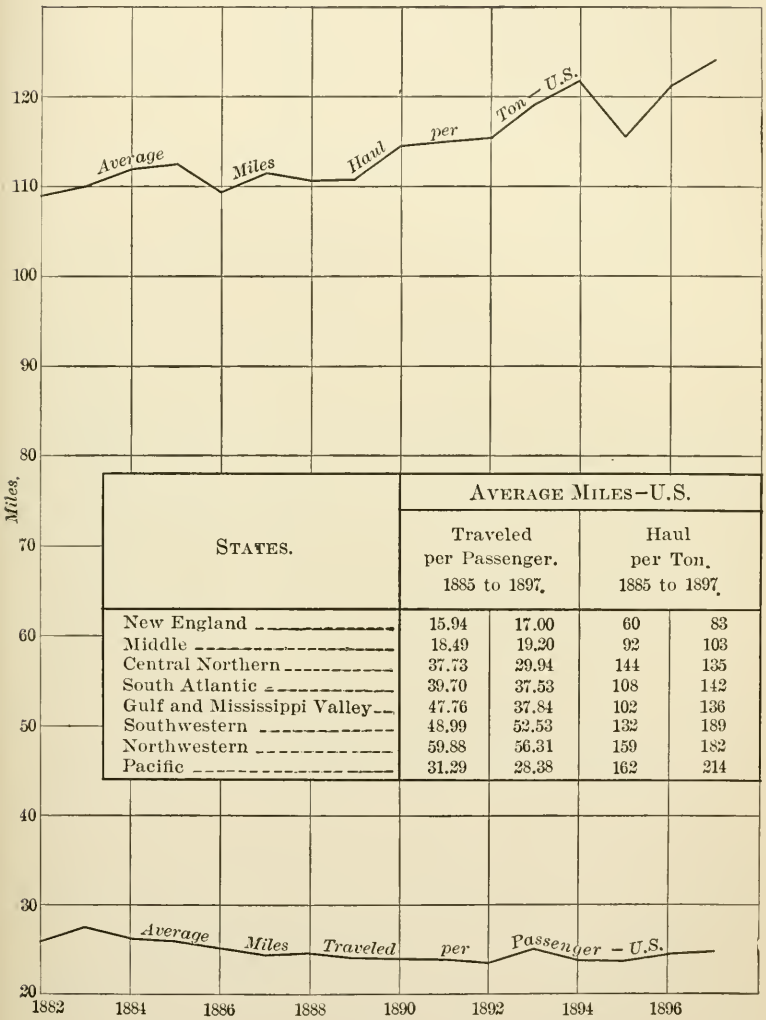

States.	Average Miles—U.S.			
	Traveled per Passenger. 1885 to 1897.		Haul per Ton. 1885 to 1897.	
New England	15.94	17.00	60	83
Middle	18.49	19.20	92	103
Central Northern	37.73	29.94	144	135
South Atlantic	39.70	37.53	108	142
Gulf and Mississippi Valley	47.76	37.84	102	136
Southwestern	48.99	52.53	132	189
Northwestern	59.88	56.31	159	182
Pacific	31.29	28.38	162	214

Fig. 13.

Mr. Davis. when compared with Curves 8 and 9, Fig. 7, showing the same items on the steam railroads of Massachusetts. Records do not give the average miles traveled per passenger on street railways. Whether or not there is a temporary increase cannot be determined; but, that in the long run it tends to decrease, there is little room to doubt, although subject to some variation.

Table No. 25 shows the increase in passengers carried per car-mile on street railways and the decrease per train-mile on steam railroads.

TABLE No. 25.

	1870.	1890.
Number of passengers carried per car-mile operated	4.91	5.51
Number of passengers carried per train-mile operated	4.71	4.30

This shows the increase in train-mileage of steam roads as compared with passengers carried, while the reverse is true of street railways.

Passenger rides per capita per annum and corresponding receipts per capita per annum have increased enormously on street railways when compared with steam railroads. The number of rides increases more rapidly than the receipts per capita; about twice as fast on steam railroads and 35% more on street railways, while the actual increase of the latter has been 204% and the former 100 per cent. In other words, the steam roads have been reducing fares to more nearly approach the low charges of street railways.

These results show most clearly how the low fares (total cost per trip, not per mile) and "leave-at-your-door" service of electric street railways have increased the passenger rides per annum and the number per car-mile so that low fares are profitable. In 1896, the rides per capita per annum were 116 for street railways and 44.5 for steam, while the receipts per capita per annum were $5.95 and $12.80, respectively.

Effect of Future Growth.—Figs. 8 and 9 are only indications of what the future has in store for railroads and railway systems in Massachusetts. The predictions have been plotted by taking two points on the street-railway curves corresponding to 1885 and 1895 and extending the straight line to 1910, while the points on steam-railroad curves were 1880 and 1890.

Those curves which when extended give a result contrary to the "characteristic sign" have been thus marked. The number of these shows the indicative nature of the information and its lack of exactness, together with the importance of cross-references to "characteristic signs" and other curves before arriving at a conclusion based on averages in any particular case. These curves might be called "pro-

TABLE No. 26.—Ratios of Comparative Statistics of Steam and Street Roads.

Item number—Table No. 16.	Items.	Ratio of Comparative Statistics, Table No. 16.							
		All electric railways in New England. Table No. 5.	Electric city railways. Table No. 4.	Electric suburban and interurban railways. Table No. 3.	Electric city, suburban and interurban railways. Tables Nos. 3 and 4.	Electric city railways. Table No. 4.	Electric suburban and interurban railways. Table No. 3.	Steam railroads of New England. Table No. 16.	Street railways of New England. Table No. 16.
1	Number of roads taken	1	0.19	0.36	0.55	1	1.82	1	1.18
3	Length	1	0.26	0.46	0.72	1	1.75	1	0.32
2	Passengers carried	1	0.21	0.25	0.47	1	1.23	1	3.96
12	" earnings	1	0.21	0.26	0.47	1	1.26	1	0.63
6	Car-miles	1	0.26	0.33	0.60	1	1.23	1	0.53 (*a*)
5	Passengers carried per mile	1	0.80	0.55	0.64	1	0.68	1	12.73
13	Passengers carried per car-mile	1	0.88	0.86	0.87	1	0.97	1	6.77
26	Passengers carried per capita per annum	1 (*d*)	0.85 (*d*)	0.47 (*d*)	0.80 (*d*)	1	0.55 (*c*)	1	3.96 (*b*)
8	Passenger earnings per mile	1	0.78	0.56	0.64	1	0.72	1	2.03
9	Passenger earnings per car-mile	1	0.84	0.88	0.87	1	1.02	1	1.08 (*a*)
11	Passenger earnings per passenger	1	0.97	1.02	0.99	1	1.05	1	0.16
32	Passenger earnings per capita per annum	1 (*d*)	0.84 (*d*)	0.47 (*d*)	0.80 (*d*)	1	0.57 (*c*)	.....	0.63 (*b*)
4	Percentage of operating expenses	1	1.03	0.97	1.00	1	0.94	1	0.89
19	Operating expenses per car-mile	1	0.85	0.79	0.84	1	0.93	1	0.87 (*a*)
17	Permanent investment per mile	1	0.99	0.76	0.85	1	0.78	1	.....
18	Capital investment per mile	1	0.90	0.70	0.77	1	0.76	1	.....

(*a*) Estimated 5 cars per train on steam railroads.
(*b*) Population served estimated to be the total in New England.
(*c*) Population served estimated to be that given in Tables Nos. 28 and 29.
(*d*) Population served estimated to be the total in New England for Column 1, and that given in Table No. 29 for Column 2; Table No. 28 for Column 3; and Tables Nos. 28 and 29 for Column 4.

jection" rather than "prediction" curves; but, nevertheless, they indicate strongly the future, with its lower percentage of earnings on capital invested, lower cost of operation, greater percentage of operating expenses and larger gross revenue coming from that enormous future growth in passenger traffic at low fares with electric propulsion.

When the population and real wealth become fixed, then such growth will cease, but only then, and will fluctuate with prosperity, but this condition is far distant for the United States.

Comparison of City and Suburban Railways.—Tables Nos. 3 and 4 have been prepared in an attempt to make the discrimination between

Mr. Davis. roads "in towns" (urban) and "between towns" (interurban). Some of those in either table may more properly belong in the other, but it is believed that the comparative results shown will be borne out in practice.

Suburban roads have been placed in Table No. 3, where their characteristics conformed to the requirements of the table. Tables Nos. 5. 6 and 7 give summaries of Tables Nos. 3 and 4.

From these tables and from Table No. 16 has been calculated Table No. 26, giving ratios of statistics of steam and electric roads. The first five items (1, 3, 2, 12 and 6) show the relative number, length and capacity of roads, for these may affect the other items in the lower part of Table No. 26 (5, 13, 26, 8, 9, 11, 32, 4, 19, 17 and 18). From this table we have Table No. 27, showing characteristic signs, when comparison is made according to the notation at the top of each column.

TABLE No. 27.

	Item.	All Electric Railways in New England to				
		Table No. 2.	Table No. 1.	Table No. 1 + 2.	Table No. 2 to Table No. 1.	Steam railroads to electric railways, Table No. 16.
5.	Passengers carried per mile	—	—	—	—	+
13.	" " " car-mile	—	—	—	—	+
26.	" " " capita per annum	—	—	—	—	+
8.	" earnings " mile	- -	—	—	+	+
9.	" " " car-mile	—	—	—	+	+
11.	" " " passenger	—	+	—	+	+
32.	" " " capita per annum	—	—	—	—	—
4.	Percentage of operating expenses	+	—	+	—	—
19.	Operating expenses per car-mile	—	—	—	—	—
17.	Permanent investment per mile	—	—	—	—	:
18.	Capital " " "	—	—	—	—	:

The characteristics brought out by these figures are the enormous number of passenger trips per capita per annum per mile of track and per car-mile, and the greater total number carried by the electric roads when compared with the steam railroads; also the larger earnings per mile of track and per car-mile on the electrics. The receipts per passenger per trip are only 16%, while the receipts per capita per annum are only 63% of the steam railroads. The steam road gets greater receipts per trip, but carries each passenger a longer distance, and has to run a disheartening number of cars or train-miles for the passengers carried. The difference is due to the short trips, high fare per passenger and car-mile, and the "leave-at-your-door"

service of electric roads, while each trip is delivered at an extremely low "total cost" to the passenger. Mr. Davis.

Suburban and interurban railways carry fewer passengers per mile, per car-mile, and per capita per annum than city railways. They earn less per mile of track, more per car-mile and per passenger carried, and less per capita per annum. They operate at a lower percentage of gross receipts and about 7% less per car-mile (Table No. 7), while they cost 25% less. A comparison of such roads with steam railroads is a more accurate one than with city railways, but for obvious reasons it could not be made here.

Effect of Population Served.—Tables Nos. 23, 28 and 29, show the approximate population served by each road in Tables Nos. 3 and 4. In locating any new line, it is essential to know, as nearly as may be, the probable business which it will secure. Upon its amount, the cost of the line completed, and the probable operating expenses, will depend the profit of the undertaking. What other similar roads have done, is the criterion upon which we base all estimates, and the three principal units used are (1) revenue per head of population (number of trips per inhabitant per annum), (2) cost per mile, and (3) operating expenses per car-mile or train-mile. The first is that in which we are now interested.

In determining the gross revenue per head of population, the usual method has been to divide the United States into arbitrary sections, such as the groups of States in Poor's Manual (see Fig. 13), and by dividing the gross receipts by the inhabitants of each division determine the average amount contributed by each individual. Of course, each person does not spend this amount, in fact, some do not contribute anything, while others expend many times the average amount each year. The passenger traffic on any road can be divided into the following general classes:

1. Those who live adjacent to the line and are compelled to ride upon it.
2. Those who are compelled to ride but who can take another line just as conveniently.
3. Those who ride for pleasure.
4. Those who come from a distant place by some other means of conveyance and then ride upon the line for any of the other three reasons.

Those of the first class are by all odds the largest in volume, and as they are the most permanent, increasing or decreasing by known amounts in stated census periods, it would be the best possible basis for our figures of receipts per head of "population served." The difficulty is to get at this "population served." An attempt has been made in these tables, in order to show how receipts would vary according to

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TABLE No. 28.—POPULATION SERVED BY ROADS IN TABLE

NAME OF RAILWAY.	Reference No.	Fig. 16. Order according to magnitude, smallest being No. 1.	Place.	Population served.	Place.	Population served.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
MAINE.						
Augusta, Hallowell & Gardiner	s 1	19	Augusta	10 527	Hallowell	3 181
Bangor, Orono & Old Town	s 2	32	Bangor	19 103	Veazie	.....
Biddeford & Saco	s 3	23	Saco	6 075	Biddeford	14 443
Lewiston & Auburn	s 4	41	Lewiston	21 701	Auburn	11 250
Portland & Cape Elizabeth	s 5	44	Portland	36 425	South Portland	.....
Portsmouth, Kittery & York	s 6	12	Kittery	2 864	York	2 444
Rockland, Thomastown & Camden	s 7	14	Thomaston	3 009	Rockland	8 174
Total and average	.....	.....	.....	.....	.....	.....
CONNECTICUT.						
Danbury & Bethel	s 8	22	Danbury	16 552	Bethel	3 401
Derby	s 9	17	Derby	5 969	Birmingham	2 000*
Fair Haven & Westville	s 10	55	New Haven	81 298	Westville	.....
Hartford, Manchester & Rockville	s 11	9	Manchester	8 222	S. Manchester	1 500*
Hartford & West Hartford	s 12	54	Hartford	53 230	W. Hartford	1 930
Meriden Electric	s 13	33	Meriden	21 652	Yalesville	.....
Norwalk St. Ry	s 14	27	Norwalk	17 747	South Norwalk	4 875
Norwalk Tramway	s 15	47	Norwalk	17 747	East Norwalk	.....
Norwich	s 16	16	Norwich	16 156	Taftville	.....
Torrington & Winchester	s 17	2	Torrington	6 048	Winsted	.....
Bristol & Plainville	s 18	6	Bristol	7 382	Plainville	1 993
Central Railway & Electric (New Britain)	s 19	18	New Britain	16 519	Berlin	2 600
Total and average	.....	.....	.....	.....	.....	.....
MASSACHUSETTS.						
Arlington & Winchester	s 20	7	Winchester	4 861	Arlington	5 629
Athol & Orange	s 21	8	Athol	6 319	Orange	4 568
Braintree & Weymouth	s 22	13	Weymouth	10 866	Braintree	4 848
Bridgewater, Whitman & Rockland	s 23	24	Bridgewater	4 249	E. Bridgewater	2 911
Brockton, Bridgewater & Taunton	s 24	53	Brockton	27 294	Bridgewater	4 249
Brockton & East Bridgewater	s 25	40	Brockton	27 294	E. Bridgewater	2 911
Dartmouth & Westport	s 26	59	Fall River	74 398	New Bedford	40 733
Dighton, Somerset & Swansea	s 27	1	Dighton	1 889	Somerset	2 106
Fitchburg & Leominster	s 28	39	Fitchburg	22 037	Leominster	7 269
Gloucester, Essex & Beverly	s 29	45	Beverly	10 821	Wenham	.....
Greenfield & Turners Falls	s 30	10	Greenfield	5 252	Montague	6 296
Haverhill & Amesbury	s 31	50	Haverhill	27 412	Merrimack	2 633
Haverhill, Georgetown & Danvers	s 32	42	Haverhill	27 412	Bradford	3 720

No. 3. Electric Suburban and Interurban Railways. Mr. Davis.

Place.	Population served.	Place.	Population served.	Place.	Population served.	Population served	Passenger trips per capita per annum of population served.
(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
Farmingdale	(a)	Gardiner	5 491			19 699	41
Orono	2 790	Old Town	5 312	Great Works		28 705	42
Old Orchard						20 518	11
						32 951	35
Cape Elizabeth	5 459					42 384	25
Portsmouth (N. H.)	9 827					15 135	53
Rockport		Camden	4 621			16 304	59
						175 676	36
						19 953	41
Ansonia	10 342					18 311	38
Montowese		Hamden	3 882	Fair Haven		86 680	56
Manchester Center		Rockville	1 000*			11 222	53
Farmington	3 179	Unionville		Plainville	1 993	60 832	6
Wallingford	6 584					28 736	50
Winnipauk						23 122	32
South Norwalk	4 875	Westport	3 715	Stamford	15 700	46 813	16
Yantic		Thamesville		Sunnyside		18 156	69
Burrville		Highland Lake				7 548	65
Forrestville		Lake Compounce				10 375	52
Plainville Center		Newington Center				19 119 (b)	74
						326 252	43 (c)
						10 490	31
						10 887	55
						15 714	63
Whitman	4 441	Rockland	5 213	Abington	4 260	21 074	25
W. Bridgewater	1 917	Raynham	1 340	Taunton	25 448	60 248	22
W. Bridgewater	1 917					32 122	11
Dartmouth	3 122	Westport	2 599			120 852	4
Swansea	1 456					5 451	142
Lunenburg	1 146					30 452	71
Hamilton		Essex	1 713	Ipswich	4 439	42 624	27
						11 548	61
Amesbury	9 798	Salisbury	1 316	Newburyport	13 947	55 106	25
Groveland	2 191	Georgetown	2 117			35 440	13

Mr. Davis.

TABLE No. 28 (*Concluded*).—POPULATION SERVED BY ROADS IN

NAME OF RAILWAY.	Reference No.	Fig. 16. Order according to magnitude, smallest being No. 1.	Place.	Population served.	Place.	Population served.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
MASSACHUSETTS (*Continued*).						
Hingham	*s* 33	3	Hingham	4 564	E. Weymouth	.....
Hoosac Valley (North Adams)	*s* 34	37	North Adams	16 074	Adams	9 213
Interstate Consolidated (R. I.)	*s* 35	46	Pawtucket (R. I.)	27 633	Seekonk	1 317
Leominster & Clinton	*s* 36	20	Leominster	7 269	Lancaster	2 201
Lowell, Lawrence & Haverhill	***s* 37**	61	Lowell	77 696	Dracut	1 996
Lowell & Suburban	***s* 38**	56	Lowell	77 696	Billerica	2 380
Lynn & Boston	***s* 39**	62	Boston	448 477	Beverly	10 821
Marlborough	*s* 40	38	Marlborough	13 805	Hudson	4 670
Millford, Hollister & Framingham	***s* 41**	21	Milford	8 780	Hopedale	1 176
Natick & Cochituate	*s* 42	28	Natick	9 118	Wayland	2 060
Newburyport & Amesbury	*s* 43	31	Newburyport	13 947	Amesbury	9 798
Newton	*s* 44	48	Newton	24 379	Waltham	18 707
Newton & Boston	*s* 45	51	Newton	24 379	Watertown	7 073
Norfolk Central (Dedham)	*s* 46	11	Dedham	7 123	Norwood	3 733
Norfolk Suburban (Hyde Park)	*s* 47	43	Dedham	7 123	Hyde Park	10 193
Northampton	*s* 48	25	Northampton	14 990	Easthampton	4 395
North Woburn	*s* 49	36	Woburn	13 499	Winchester	4 861
Providence & Taunton	*s* 50	60	Providence (R. I.)	132 146	Taunton	25 448
Quincy & Boston	*s* 51	49	Quincy	16 723	Weymouth	10 866
Reading & Lowell	*s* 52	4	Reading	4 088	Wilmington	1 213
Rockland & Abington	*s* 53	29	Abington	4 260	Rockland	5 213
Rockport	*s* 54	34	Gloucester	24 651	Rockport	4 087
Southbridge & Sturbridge	*s* 55	5	Southbridge	7 655	Sturbridge	2 074
South Middlesex (Natick)	*s* 56	30	Natick	9 118	Sherborn	1 381
Taunton & Brockton	*s* 57	52	Brockton	27 294	Taunton	25 448
Wakefield & Stoneham	*s* 58	35	Wakefield	6 982	Stoneham	6 155
Warren, Brookfield & Spencer	***s* 59**	26	Warren	4 681	W. Warren	.....
Worcester & Blackstone Valley	*s* 60	15	Millbury	4 428	Sutton	3 180
Worcester & Marlborough	*s* 61	58	Worcester	84 655	Shrewsbury	1 449
Worcester & Suburban	*s* 62	57	Worcester	84 655	Millbury	4 428
Total and average	.....	.....	.....	.....	.....	.....
Grand total and average	.....	.....	.....	.....	.....	.....

***s* 2.** Also—

Stillwater	....

***s* 15.** Also—

Roton Point	....
Rowayton	....
Darien	2 276
Noroton	....
Dorlon's Point	....
	4 276

***s* 29.** Also—

Gloucester	24 651

***s* 37.** Also—

Haverhill	27 412
Groveland	2 191
W. Newbury	1 769
Newburyport	13 947
N. Andover	3 742
	49 061

***s* 38.** Also—

Tewksbury	2 515

***s* 39.** Also—

Lynn	55 727
Malden	23 031
Marblehead	8 202
Melrose	8 519
Peabody	10 158
Revere	5 668
Salem	30 801
Saugus	3 673
Stoneham	6 155
Swampscott	3 198
Wenham	....
Woburn	13 499
Hamilton	....
	169 631

TABLE No. 3. ELECTRIC SUBURBAN AND INTERURBAN RAILWAYS. Mr. Davis.

Place.	Population served.	Place.	Population served.	Place.	Population served.	Population served.	Passenger trips per capita per annum of population served.
(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
N. Weymouth		Hull	2 000*			7 564	119
Williamtown	4 221					29 508	52
Attleborough	7 577	N. Attleborough	6 727	Wrentham	2 566	45 820	56
Clinton	10 424					19 894	43
Methuen	4 814	Lawrence	44 654	Andover	6 142	184 363	48
Chelmsford	2 695	Dracut	1 996	Tyngsborough		87 782	87
Chelsea	27 909	Danvers	7 454	Everett	11 068	675 360	43
Southborough	2 114	Framingham	9 239			29 828	25
Holliston	2 619	Ashland	2 532	S. Framingham		19 926	83
Wellesley	3 600	Framingham	9 239			24 017	45
Merrimac	2 633	Newbury	1 427			27 805	40
Watertown	7 073					50 159	36
Needham	3 035	Boston ($\frac{1}{26}$)	22 423			56 910	19
Walpole	2 604					13 460	52
Boston ($\frac{1}{26}$)	22 423					39 739	48
Williamsburg	2 057					21 442	84
Medford	11 079					29 439	27
						157 594	4
Milton	4 278	Boston ($\frac{1}{26}$)	22 423			54 290	49
Billerica	2 380					7 681	43
Weymouth	10 866	Whitman	4 441			24 780	61
						28 738	19
						9 729	56
Framingham	9 239	Ashland	2 532	Hopkinton	4 088	26 358	43
Easton	4 493	Raynham	1 340			58 575	16
Reading	4 088	Saugus	3 673	Melrose	8 519	29 417	38
Brookfield	3 352	W. Brookfield		N. Brookfield	3 871	21 651	44
Grafton	5 002	Northbridge	4 603			17 213	16
Northborough	1 952	Westborough	5 195	Marlborough	13 805	107 056	11
Leicester	3 120	Spencer	8 747			100 950	32
						1 827 517 (*d*)	49
						2 352 765	43

§ 41. Also—
Bellingham.... 1 334
Medway..... 2 985
4 319

§ 59. Also—
Spencer..... 8 747

* Estimated.

(*a*) Where populations are omitted, the census made no return; for approximation 500 inhabitants are assumed in each case.

(*b*) This total is 24 615 less than the column foots up, due to repetition of several cities in the table.

(*c*) If *s* 12 were omitted, this average would be 51.

(*d*) This total is 631 546 less than the column foots up, due to repetition of several cities in the table.

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TABLE No. 29.—POPULATION SERVED BY ROADS

NAME OF RAILWAY.	Reference No.	Order according to magnitude, smallest being No. 1 (Fig. 17)	Place.	Population served.	Place.
MAINE.					
Bangor St. Ry.	C 1	15	Bangor	19 103	Brewer
Bath St. Ry.	C 2	3	Bath	8 723	
Calais St. Ry.	C 3	6	Calais	7 290	St. Stephens (N.B).
Portland R. R.	C 4	25	Portland	36 425	Deering
Total and average	...	.....	.....	.....	.....
CONNECTICUT.					
Bridgeport Traction	C 5	27	Bridgeport	48 866	Southport
Hartford St. Ry.	C 6	33	Hartford	53 230	Withersfield
Middletown St. Ry.	C 7	8	Middletown	9 013	Portland
New Haven St. Ry.	C 8	31	New Haven	81 298	East Haven
New London St. Ry.	C 9	9	New London	13 757	Ocean Beach
Stamford St. Ry.	C 10	10	Stamford	15 700	
Winchester Ave. (N. Haven)	C 11	34	New Haven	81 298	Savin Rock
Waterbury Traction	C 12	22	Waterbury	28 648	Waterville
Total and average	...	.....	.....	.....	.....
MASSACHUSETTS.					
Boston Elevated	C 13	35	Boston	448 477	Cambridge
Braintree	C 14	20	Braintree	4 848	Quincy
Brockton	C 15	26	Brockton	27 294	Whitman
Commonwealth Ave. (Newton)	C 16	17	Newton	24 379	
Framingham Union	C 17	5	Framingham	9 239	
Gardner Electric	C 18	2	Gardner	8 424	
Globe (Fall River)	C 19	30	Fall River	74 398	
Gloucester	C 20	18	Gloucester	24 651	
Holyoke	C 21	29	Holyoke	35 637	Chicopee
Pittsfield Electric	C 22	13	Pittsfield	17 281	Dalton
Plymouth & Kingston	C 23	4	Plymouth	7 314	Kingston
Springfield	C 24	28	Springfield	44 179	W. Springfield
Taunton	C 25	19	Taunton	25 448	
Union (New Bedford)	C 26	24	New Bedford	40 733	Fairhaven
Wellesley & Boston	C 27	16	Newton	24 379	
West Roxbury & Roslindale	C 28	21	Boston ($\frac{1}{10}$)	22 423	Dedham
Worcester Consolidated	C 29	32	Worcester	84 655	
Woronoco	C 30	7	Westfield	9 805	
Total and average	....	.....	.....	.....	.....
" " (C 13 omitted)	....	.....	.....	.....	.....
NEW HAMPSHIRE.					
Union St. Ry.	C 31	12	Dover	12 790	Somersworth
Nashua St. Ry.	C 32	14	Nashua	19 311	Hudson
Concord St. Ry.	C 33	11	Concord	17 004	
Manchester St. Ry.	C 34	23	Manchester	44 126	
Laconia St. Ry.	C 35	1	Laconia	6 143	
Total and average					
Grand total and average (C 13 omitted)					

C 5. Also East Bridgeport .....
C 6. Also East Windsor 2 890
Hockanum .....
N. Glastonbury } ...
S. Glastonbury } 3 457
Burnside .....
Newington .....
New Britain 16 519
S. Windsor 1 736
E. Hartford 4 455
30 557

C 8. Also Centerville .....
C 11. Also Milford 3 811
Orange 4 537
8 348

C 13. Also Newton 24 379
Somerville 40 152
Arlington 5 629
Brookline 12 103
Watertown 7 073
Medford 11 079
Malden 23 031
123 446

IN TABLE No. 4. ELECTRIC CITY RAILWAYS. Mr. Davis.

Population served.	Place.	Population served.	Place.	Population served.	Population served.	Passenger trips per capita per annum of population served.
4 193	.....	.....	.....	.....	23 296	65
.....	.....	.....	.....	.....	8 723	51
2 000?	Hilltown (N. B.)	..... (*a*)	.....	.....	9 790	49
5 353	Westbrook.....	6 632	.....	.....	48 410	112
.....	.....	.....	.....	.....	90 219	87
.....	Westport	3 715	Fairfield.....	3 868	57 449	71
2 271	Windsor	2 954	W. Hartford...	1 930	90 942	98
4 687	.....	.....	.....	.....	13 700	29
.....	Morris Cove.....	.....	Westville.....	.....	83 298	37
.....	.....	.....	.....	.....	14 257	46
.....	.....	.....	.....	.....	15 700	39
.....	City Point.....	.....	W. Haven.....	.....	91 146	48
.....	Naugatuck.....	6 218	.....	.....	35 366	71
.....	.....	.....	.....	.....	401 858	61
70 028	Chelsea.....	27 909	Everett.....	11 068	680 928	266
16 723	Randolph	3 946	Holbrook.....	2 474	27 991	34
4 441	Stoughton.....	4 852	Holbrook	2 474	48 884	139
.....	.....	.....	.....	.....	24 379	59
.....	.....	.....	.....	.....	9 239	68
.....	.....	.....	.....	.....	8 424	40
.....	.....	.....	.....	.....	74 398	89
.....	.....	.....	.....	.....	24 651	55
14 050	Northampton.....	14 990	S. Hadley.....	4 261	68 938	61
2 885	.....	.....	.....	.....	20 166	65
1 659	.....	.....	.....	.....	8 973	77
5 077	Chicopee	14 050	Longmeadow..	2 183	65 489	177
.....	.....	.....	.....	.....	25 448	44
2 919	Dartmouth.....	3 122	.....	.....	46 774	80
.....	.....	.....	.....	.....	24 379	29
7 123	.....	.....	.....	.....	29 546	49
.....	.....	.....	.....	.....	84 655	125
.....	.....	.....	.....	.....	9 805	48
.....	.....	.....	.....	.....	1 215 794 (*b*)	193
.....	.....	.....	.....	.....	581 668 (*c*)	93 (*d*)
6 207	.....	.....	.....	.....	18 997	31
1 022	.....	.....	.....	.....	20 403	46
.....	.....	.....	.....	.....	17 004	57
.....	.....	.....	.....	.....	44 126	74
.....	.....	.....	.....	.....	6 143	37
.....					106 673	56
.....					1 180 418	78

C 15. Also Easton..... 4 493
Randolph..... 3 946
Avon..... 1 384
9 823

(*a*) Where populations are omitted, the census made no return; for approximation, 500 inhabitants are assumed in each case.
(*b*) This total is 67 272 less than the column foots up, due to repetition of several cities in the table.
(*c*) Population served, omitting *C* 13.
(*d*) Rides per capita per annum of population served, omitting *C* 13

Mr. Davis. the basis of estimates, and how much care must be taken not to be led astray in any given case. If we assume that the steam roads of the New England States serve the entire population, we get gross receipts per head, equal to $7.32, and they undoubtedly do serve this population, but if we were about to build the Boston and Albany Railroad (assuming it did not exist), could we properly use this figure, multiply by the total population and say that would be the probable receipts; or should we take the population within the limits of division (1) "those who live adjacent to the line and are compelled to ride upon it" and multiply by some higher figure, determined from other similar cases? The latter appeals to one as likely to produce much more accurate results. Table No. 16 gives the various amounts per head per annum for various assumed "populations served," from urban communities having, according to the United States census, populations of 8 000 and upward, to and including towns of 1 000 and upward. These divisions of population are not advanced as the proper ones, for they are not, but to bring out clearly what has been said before.

Turning to Tables Nos. 28 and 29, we see great fluctuations in the rides per capita per annum; this is partly due to many causes, such as population, density, distribution, etc., etc., but also due to inaccuracies in determining "population served." From the tables it is seen that the entire populations of towns are repeated for several roads and are considered as being served by and serving these roads; this is necessary unless we can divide the territory, which would be most difficult. If, however, it were done, and it can be, the results would be most interesting and instructive. By the divisions we have made, it is shown clearly that suburban and interurban roads either do not serve as great a population as the aggregate of those forming their terminals and through which they pass, or else the receipts per capita per annum are less from those they do serve. As a matter of fact both propositions are probably true. Our average of 78 in Table No. 29 would have been increased materially had road *C* 13 (Boston Elevated) been included with its 266 rides per capita per annum, but the result would not have been as accurate for application to other problems.

Effect of Competition.—One often hears of the competition which electric parallels have brought to our steam-railroad systems. This has been exaggerated greatly, for most of the traffic of electric railways did not exist until created by low "total cost" and frequent and quick service, although, in certain isolated cases, the building of electric parallels has temporarily drawn away traffic from steam railroads, only to be recovered as the total volume naturally increased. This fluctuation and recovery in traffic, on parallels which changed motive powers, has been shown clearly in the building of elevated and street railways in New York City. The Third Avenue Elevated so decreased the traffic on

Mr. Davis.

the horse surface road as to cause alarm to the stockholders; the conversion of the horse railway to a cable road decreased the travel on the elevated, which was subsequently more than recovered.

Fig. 14 gives a good example of the decrease in traffic on the Manhattan Elevated Railway in New York City, operated by steam locomotives, due to the increase in speed and frequency of service on the Metropolitan Street Railway in its changes from horse traction to cable and electric. The loss on the elevated road from 1893 to 1897 was approximately 40 000 000 passengers—part of which was due to the financial depression throughout the United States, as indicated by the "dip" in all curves on Fig. 15; how much this amounts to it is impossible to determine and most difficult to estimate, but an approximation can be made from the retardation of increase shown in the curve on Fig. 15, giving passenger trips on all electric roads in Massachusetts. Projecting the curve by connecting 1893 with 1897 it would indicate a natural proportional increase in 1894 of 17 000 000 more passengers than actually took place, which represents approximately

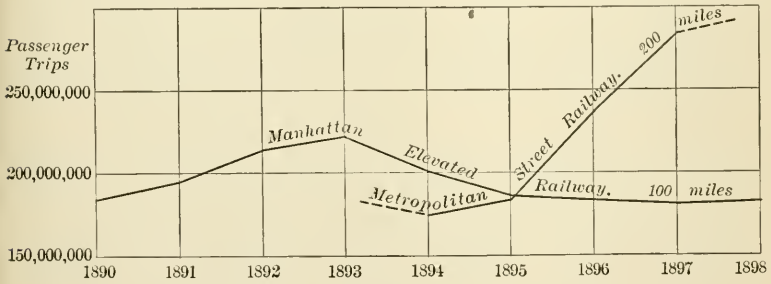

FIG. 14.

the retardation due to the financial depression, or about 7.7% of the total traffic. If we assume the same loss, from the same cause, on the elevated railways, about 17 000 000 of the above 40 000 000 loss is accounted for by the industrial depression, leaving 23 000 000 loss due to competition. During the same period the Metropolitan Street Railway gained about 110 000 000 passengers, or nearly five times as many as the elevated roads lost. The mileage of both roads remained constant, or nearly so, as in the Metropolitan curve are included, for the years taken, all roads now owned or operated by this company. These years also cover the change in motive power of this system. This increase in traffic on the street railways of New York undoubtedly comes largely from increase in speed, better physical conditions, such as track, cars, lighting, heating, cleanliness, open cars, etc., etc.

The elevated railways are operated under the same general conditions as exist on street railways, although the units are heavier, but they are very frequent; of course, they have the disadvantage of being

Mr. Davis.

TABLE No. 30.—Effect of Competing Electric Interurban Parallels to Steam Railroads.

Localities Connected.	Loss due to trolley parallels, as claimed by V. P. Hall of N. Y., N. H. & H. R. R., before Railroad Committee, State Legislature of Connecticut (*a*).	Approximate distance in miles from Railroad Committee Map.	Trips per day as given by time table of N. Y., N. H. & H. R. R.	Trips per day of trolley roads.	Number of passengers carried by Electric Railway System in and between these towns, part of which traveled between them, 1894.
Norwalk—Rowayton	50%	4.75	27	84	956 241
Bridgeport—Stratford	$35 per day.	3.00	36	84	4 659 322 (Est.)
Bridgeport—Southport	80%	5.50	23	63	
Waterbury—Naugatuck	90%	6.00	13	69	2 624 421
Wallingford—Meriden	30%	5.50	17	30	2 001 347
Birmingham—Ansonia	90%	3.00	16	112	1 033 977
Winnepank—S. Norwalk	(*b*) 50%	3.00	....	....	1 090 263
					12 365 571

(*a*) Total loss to N. Y., N. H. & H. R. R. from all parallel trolley roads in the State of Connecticut = $4 000 per month = $48 000 per annum, or ⅛ of 1% loss on total passenger income of $12 971 000 in 1894, as shown by Railroad Committee Reports.

(*b*) 64 passengers were carried on N. Y., N. H. & H. R. R. in the month of December, 1893, and 9 in the same month of 1894, or a total loss of 780 passengers per annum, at a possible maximum of 15 cents = $117.

confined to what might be called "trunk" lines without feeders. These frequent units are now operated by steam locomotives, but a change to electric motors is about to take place. This change is not warranted by any decrease in operating expenses which will take place, either actual or sufficient to offset the interest on the additional investment, although in the first years of electric operation figures will no doubt be produced which will appear to indicate such a result, as in two cases already cited. Nevertheless, the change, if made, will be a profitable one from the natural and induced increase in the traffic of the future, just as the large investments of our steam railroads in improved terminals, track, rolling stock and stations have been justified. Similar results will be the inducements for a change of operative methods on parts of our steam railroad systems, which change necessitates a change in motive power.

Table No. 30 has been prepared from a speech by Mr. Edwin B. Gager before the Railroad Committee of the State Legislature at Hartford, Conn., March 22d, 1895.* For many years bitter warfare has been waged against interurban electric railways by the Consolidated System (New York, New Haven and Hartford Railroad Company) resulting in the electric parallel law, where "public convenience and necessity" must be demonstrated to the satisfaction of the Superior

* Those who are interested can obtain a reprint of Mr. Gager's speech by addressing him at Derby, Conn.

Mr. Davis.

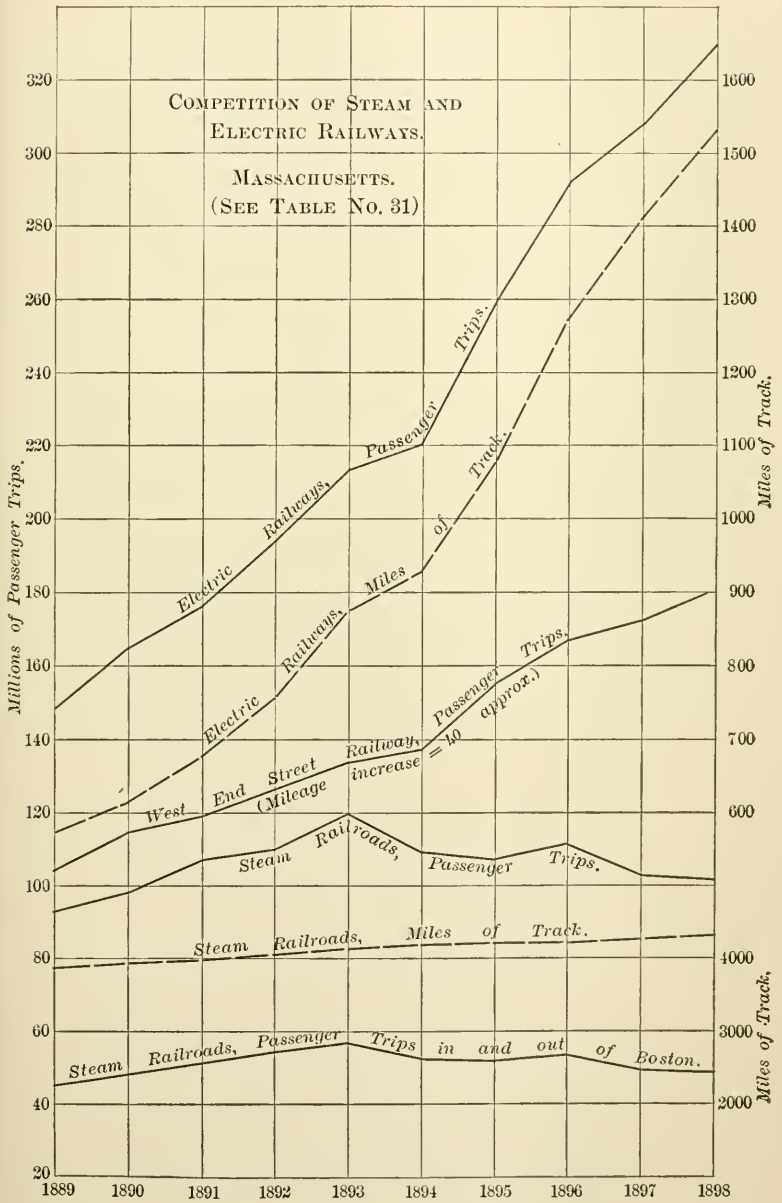

Fig. 15.

Mr. Davis. Court, before an electric railway can be built between two points already connected by a steam railroad. More unwise legislation against a natural progress, which would also benefit those whose influence created it, can scarcely be imagined. It is fair to assume that in this controversy—for the street railways naturally opposed such legislation—both sides produced the strongest arguments in support of their respective contentions; the Consolidated presenting losses of traffic, while the street railways insisted that their passenger travel was mostly an induced one, which did not and could not exist under steam-railroad conditions and operative methods.

An examination of Table No. 30 shows conclusively how the steam railroads convicted themselves. The Consolidated System only claimed a total loss of $4 000 per month, or $48 000 per annum on the entire system, being about $\frac{1}{3}$ of 1% of their gross passenger revenue. If the average fare were 10 cents, this would mean a total loss of 480 000 passengers per annum out of a total of 44 448 324, or 1.1%; but 1894 was the year of financial depression, when the steam railroads of Massachusetts lost 8.3% of their former passenger traffic, so that only part of this loss on the Consolidated was due to trolley parallels. While the total loss to the Consolidated was given by its officers, all the towns between which it occurred were not stated, so that in Table No. 30 the 12 365 571 passenger trips, between and in a few of these towns, is only part of the total passenger traffic of the street railways serving all localities where such loss took place. Whether this figure should be increased by 50 to 100%, or more, we cannot say, but, in any case, the data are sufficient to show the large induced traffic of street railways; or, in other words, systems which give low fares, frequent service, short total time consumed in round trip and a "leave-at-your-door" service.

To emphasize the fact of what might be called a "dormant traffic," which can become an "induced traffic" under proper operative conditions, and to bring out this fact more clearly, Fig. 15 and Table No. 31 have been prepared, and these again show what a small part of the traffic of street railways has come from the losses of steam railroads, and also, that a large part of this loss has been wrongly attributed to electric parallel competition.

The passenger traffic on steam railroads in Massachusetts has increased constantly from year to year since 1870, except during two periods, both of which coincide with industrial depressions; this is shown by Curve 3, Fig. 7, where the loss began in 1873 and 1893. Short-haul passenger traffic is but little affected by financial conditions, when compared with the effect on long-haul traffic; this is due mainly to necessity being the basis of short-haul passenger business, or, in other words, it is composed mostly of commuters or suburban and interurban travel. Curve 2, Fig. 6, corroborates this position,

for we see that the passenger trips on street railways (short hauls, from necessity, as the controlling factors in volume of traffic on these systems), increased after 1874 and 1893, although less rapidly than in the year previous, but there was no actual decrease. This curve brings out another interesting fact, namely, the quicker recovery of electric roads and their more rapid increase in passenger traffic than when operated by horse-power, again supporting our position. In Massachusetts, electric railway mileage has increased along with the passenger traffic (1889 to 1898) 163% in the former to 123% in the latter, or, approximately, each has kept pace with the other. In other words, these railways have been built where traffic did not previously exist, nor could it be produced by the steam railroads under existing conditions; it has been "induced" by the character of the electric roads and their operative methods. Steam railroad mileage has only increased 10% in Massachusetts during the same period, and passenger traffic 9.6% (net). Mr. Davis.

TABLE No. 31.—Comparative Loss on Steam Railroads and Gain on Electric Railways—Massachusetts.—(See Fig. 15).

Year.	Loss in passenger trips per annum on all steam railroads in Massachusetts.	Loss in passenger trips per annum on steam railroads in and out of Boston.	Loss in passenger trips per annum on steam railroads in Massachusetts omitting those in and out of Boston. Column 1 minus Column 2.	Gain in passenger trips per annum on all street railways in Massachusetts.	Amounts in Column 4 divided by those in Column 1.	Gain in passenger trips per annum on West End Street Railway system in and out of Boston.	Amounts in Column 6 divided by those in Column 2.
1889 to 1893..	26 250 648	11 406 434	14 844 214	65 362 606	2.4	29 620 468	2.6
	(Increase)	(Increase)	(Increase)	(Increase)	.....	.....	.....
1894.....	10 345 763	3 824 940	6 520 823	6 912 090	0.65	3 164 831	0.8
1895.....	1 577 836	740 680	837 156	39 330 209	24.9	18 208 057	24.5
1896.....	(Increase)	(Increase)	(Increase)	(Increase)	.....	(Increase)	.....
1897.....	8 885 161	3 808 480	5 076 681	16 325 281	1.8	5 692 225	1.4
1898.....	803 168	723 748	79 420	22 205 405	27.6	8 766 782	12.1
	21 611 928	9 097 848	12 514 080	.....	.....	.....	.....
1896 increase..	3 772 703	1 483 774	2 288 929	32 564 635	8.6	11 630 782	7.8
Net decrease 1894 to 1898.	17 839 225	7 614 074	10 225 151	117 327 620	6.5	47 457 677	6.2

Turning to Table No. 31 and Fig. 15, we see that the net loss of passenger traffic on all the steam railroads of Massachusetts, from 1893 to 1898, was 17 839 225; but during the same years the street railways increased 117 327 620, or 6.5 times the loss of the steam railroads. If 7.7% of the traffic was lost, due to financial conditions (as estimated heretofore), then only 8 600 000 passenger trips were lost to

Mr. Davis. the steam roads of the State from trolley competition. If the average fare lost was 10 cents (when secured by the street railway the fare would be halved or even less for them), the total amounted to $860 000; or 2.4% on the gross passenger earnings and 1.1% on the gross earnings of the steam railroads; this would only amount to about 1% of the total net earnings. Again, the net loss to steam railroads in and out of Boston, from 1893 to 1898, was 7 614 074, while the West End Street Railway (controlling practically all street railways in and out of Boston), increased 47 457 677 or 6.2 times the loss of the steam roads. Applying the same argument, only 3 225 000 passengers were lost by the steam roads in and out of Boston, due to this competition. This would not be fair, however, for this traffic is made up more largely of commuters than long-haul passengers. Assume it at 5 000 000, which is undoubtedly too high, and with an average commutation rate of 7 cents, the loss is only $350 000 at the maximum. Furthermore, the greatest gains of the street railways throughout the State, and the West End Street Railway alone, were in 1895 and 1898, when the steam roads lost the least. All these data point to the conclusion, already stated, that competition does not take place to the extent usually believed; while an "induced" traffic is created by the low fares, frequent, quick and "leave-at-your-door" service, rendered possible by the physical characteristics and operative methods of the electric roads.

Fig. 16 is from Table No. 28. Populations are arranged in the order of magnitude as abscissas, beginning with the smallest served, while the passengers per capita per annum are ordinates. The resulting curve shows no well-defined law.

Fig. 17 is the same from Table No. 29, and indicates plainly that as populations increase, the rides per capita per annum also increase.

Fig. 18 is made from towns arranged in the order of their density of population. The curve shows no well-defined law.

The determination of the desirability or undesirability of substituting electric traction for steam on parts of some American railroad systems is a matter of constantly increasing importance; on many projected lines the question whether steam or electricity shall be used is a problem which must now be solved, although in the past there was no question as to which should be adopted. We will probably see during the next ten years a large increase in the substitution of electricity for steam on railroads. This change is foreshadowed by the present electrical equipment of elevated railways in Chicago; by the overhead-trolley and third-rail experiments of the New York, New Haven & Hartford Railroad Company; by the operation of several branch lines of the Pennsylvania Railroad Company with electric motors; and by the present active discussions, among steam-railroad engineers and managers, of the details of first cost, operating expenses, and methods of electric traction.

Mr. Davis.

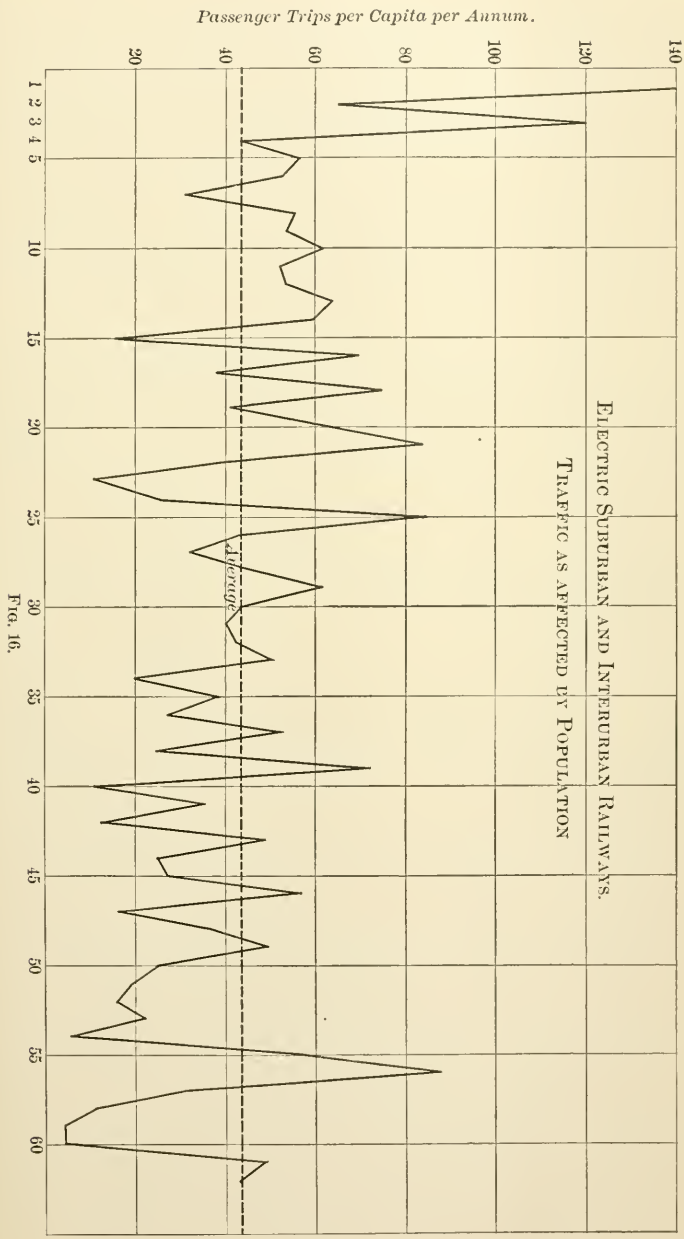

Fig. 16.

Mr. Davis.

If a change is contemplated, steam-railroad managers must avoid making the mistake which took place in the change from horse traction to electric traction—namely, of trying to reduce the first cost of changing by the use of old methods, material, and equipment, which, although entirely suited to the old system, proved most unsatisfactory under the new conditions. The old equipment partly made over will not do. There must be new trucks, and new and lighter car bodies, hung lower for greater ease of entrance and departure. Old methods of operation must be discarded, and new ones substituted. A change of system may necessitate additional tracks, which should be provided even at large cost. The nearer the approach to the "leave-at-your-door" service, the greater the success.

Let us consider some examples where it is probable that electricity could be substituted with profit, remembering that more thorough investigation might show, in this or that case, that the change is undesirable.

Suburban service of the Pennsylvania Railroad, out of Philadelphia, as follows:

P., W. & B.	Division to	Wilmington.
Main Line	"	" Paoli.
Germantown & Chestnut Hill	"	" Chestnut Hill.
Reading	"	" Norristown.
U. R. Rs. of N. J.	"	" Tacony.
West Chester	"	" West Chester.

By the use of multiphase currents and rotary-transformer sub-stations, power houses could be located along the Delaware and Schuylkill rivers; from these sub-stations direct current at 500 volts could be fed into the trolley or third-rail line. The equipment should be entirely new; cars of the same seating capacity, but lower and of lighter construction, should be used; the bridge over the Schuylkill River into Broad Street Station should be widened by four additional tracks; main line and U. R. Rs. of N. J., by two tracks, each; P. W. & B. should have a total of four tracks; Germantown & Chestnut Hill, Reading, and West Chester should each have a total of three tracks; all tracks should be equipped electrically, although some of them would be used by steam locomotives hauling freight and through passenger trains, and some or all, at times, by both systems. Two tracks should be laid in the street from Broad Street Station to the Schuylkill Bridge coming to railroad grade at that point; the cars running over these tracks should operate from the Delaware River out Market Street, by trackage arrangement with the street railway or by special franchise, the street being wide enough for four tracks; they should be run as any street-railway line is run to-day, and continue out on the extra tracks, provided above, as far

Mr. Davis.

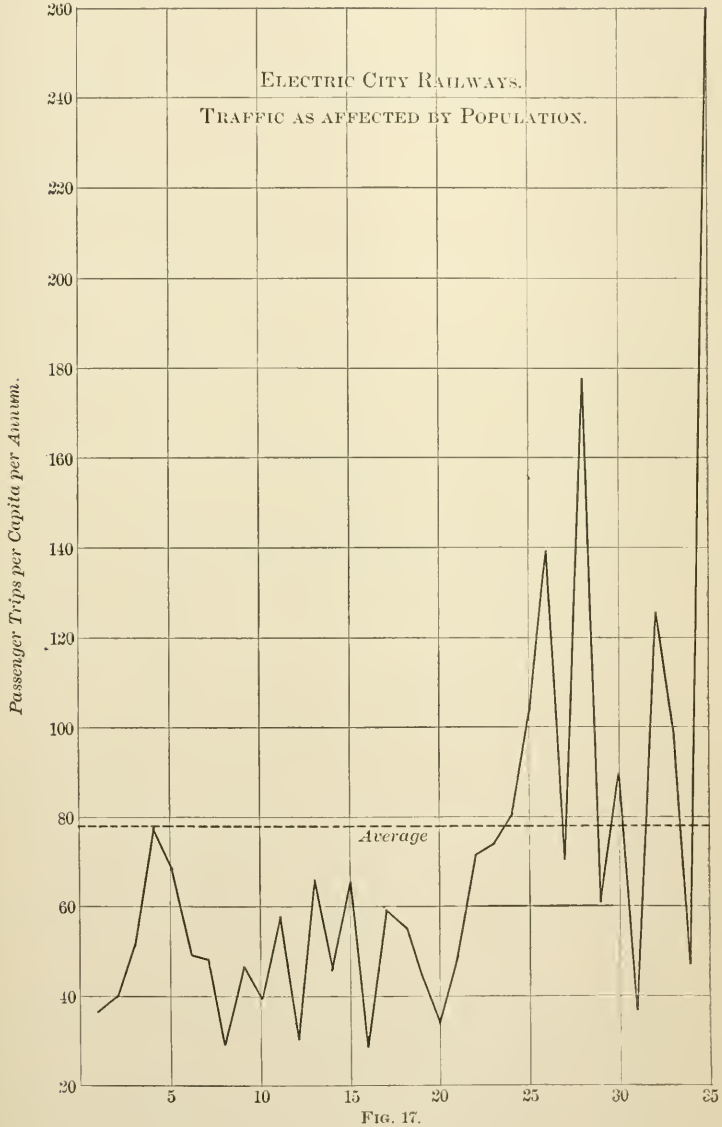

Fig. 17.

Mr. Davis. as circumstances might warrant, and should be extended as travel required; right-of-way should be prepared, so that passengers could leave or enter these cars at any point along the line. Local stations should be provided, in some cases much closer than at present, at which passengers, on the cars mentioned above, could change to electric cars stopping only at said stations, or to express electric trains stopping at fewer points, all being done by the payment of a single fare. It will be seen that this is a combination of the street railway on the outside tracks with the local and express service of a steam road on the inside tracks, while the through locomotive trains operate on the middle tracks. The proposition is a radical one, is advanced as such, and will bear close investigation and study; the expense would be enormous, but the gross receipts from such a system of street railways, accumulating passengers for the rapid-transit system connected so intimately with it, would also be enormous—in fact, the writer believes, far in excess of anything yet accomplished in the transportation of passengers.

The same treatment of the following terminals, with modifications to suit each case, would be worth studying:

New York, New Haven and Hartford Railroad, out of Boston.
Boston and Maine, out of Boston.
Philadelphia and Reading, out of Philadelphia.
Harlem, New York Central and Hudson River, and New York, New Haven and Hartford Railroad Companies, out of New York City.
Central Railroad Company of New Jersey, out of Jersey City.
Illinois Central Railroad, out of Chicago, etc., etc.

The above is, of course, only a suggestion of details.

As an example of interurban traffic, we can take Ansonia and New Haven, Conn. When the steam railroad owns the systems of street railways in both towns, their cars will pick up passengers at either center, will pass onto the present steam tracks on the right-of-way of the New Haven and Derby Railroad (New York, New Haven and Hartford Railroad Company), run at high speed without stops to the other center, pass onto the local street railway tracks there and distribute its passengers where they desire, all for one fare. Such a system operated by electric motors would be a financial success, where a line like the third-rail between Hartford and New Britain is a failure in the true sense. Many other similar examples might be given, but this indicates the future of our steam railroads.

It must not be inferred that all or any of the cases suggested above, if thoroughly investigated, would prove the expense warranted, especially that of laying additional tracks. It is, however, the writer's judgment that in most cases additional tracks would be necessary to

make the system profitable and able to handle the "induced" traffic. Mr. Davis.
Emphasis must be placed upon the importance of radical changes in present steam-railroad methods to make the change to electricity pay. Electric traction in connection with existing tracks and equipment, and under existing operative methods, will result in heavy loss and ultimate abandonment of the change—possibly even in disaster.

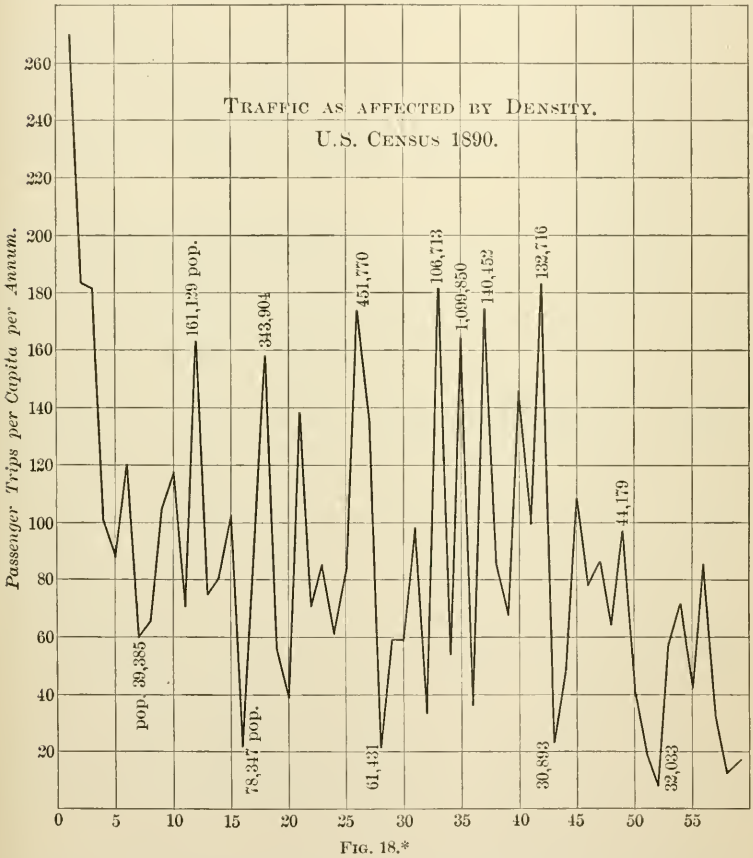

Fig. 18.*

The argument has thus far been confined, for reasons stated, to suburban and interurban short hauls, and the writer feels convinced from his investigation of the subject that the adoption of electricity as a mode of operation, with the additional change to the prevailing

* The data for the diagram, Fig. 18, are taken from Table No. 39 of a paper by Charles H. Davis, M. Am. Soc. C. E., entitled "Facts and Figures Interesting to Electric Railway Men."

Mr. Davis. methods of our present street railways giving nearly the same class of service (combined or not with steam locomotives as the case may require), will result to the steam roads in an enormous increase of the total passengers carried, passengers per car-mile and rate received per mile, and will shorten the length each passenger is hauled; and, by thus increasing the gross receipts, will more than pay for the additional investment. If this cannot be done in any given case, then it will not pay to make the change.

In conclusion, the writer wishes to quote from the final paragraphs of a paper presented by him to the American Institute of Electrical Engineers,* wherein was expressed the convictions given in this discussion, and which he sees no reason to change at this writing. This was prior to the third-rail experiments of the New York, New Haven and Hartford Railroad.

"The question whether the gross receipts of a given road will be affected by the use of one or the other power under discussion is a most interesting one. Experience shows that where an electric road has paralleled a steam road it has taken most of the latter's business at first, but less as time went on; and that it created a demand for intercommunication which had never existed before—the bulk of the passenger travel coming from this cause. This is interestingly shown in the arguments of Judge Hall (Vice-President of the New York, New Haven and Hartford Railroad) and Judge Gager, of Connecticut, before the Legislature of the State at its last session. This, of course, refers to passenger receipts only. Freight receipts would not be affected by the use of one power or the other, they increasing only as the country grows and rates fall, together with better facilities. Receipts from express and mails might be materially increased by the use of electric traction when giving more frequent service. It appears that the close headway and 'leave-at-your-door' service of electric roads are the main reasons for their induced travel. The question of how much more the travel would be increased by the use of electric traction and frequent service is problematical, for the 'leave-at-your-door' service is wanting in steam railways as they are, but why not change them? If this could be done, past experience and data would give a good basis from which to estimate future results.

"The conclusion one arrives at is that for long lines, infrequent service, where freight is a large proportion of the business, and where centers of population are far apart, the steam locomotive is the only paying method of to day, as the first cost will be less, as well as total expenses. The writer has had several opportunities of determining these facts. What development may bring to electric traction in the far future cannot be foretold.

"The writer believes that electric traction will be profitable to steam railway systems when some or all of the following conditions are fulfilled, depending upon the special problem to be solved:

"1. Steam railway managers must avoid making the mistake which took place in the change from horse traction to electric traction, namely, of trying to reduce the first cost of changing by use of old methods, material and equipments, which, although entirely suited to

* Meeting of October 21st, 1896.

the old system, proved most unsuitable under the new conditions. The tendency is to repeat this mistake, and too much stress cannot be laid upon avoiding it. The old equipment partly made over, the old method of operating, etc., will not bring success in the use of electric traction; and if followed from necessity would indicate the strongest argument against the change. Mr. Davis.

"2. Long distance, heavy trains and infrequent service, if a necessity, will prevent electric traction being profitable. Therefore, where gross receipts can be increased by light trains and frequent service, and thus decrease expenses, as compared to steam locomotives, electric traction will prove profitable. One of the best examples of how this could be applied is found in the suburban service of the Pennsylvania Railroad out of Philadelphia. It is, of course, understood that electric cars can be operated over the same tracks as trains drawn by steam locomotives; a change of system requiring more frequent service for success might necessitate one or more additional tracks, which in some cases would delay the time when a change would be advisable.

"3. Steam railways, where the second condition is fulfilled, can better the results where they operate part of the system on the 'leave-at-your-door' plan. This suggestion may seem to some a radical departure, but I commend it to the careful thought of those interested."

The writer's final conclusion is that electric traction can be profitably used:

(1) Where units are so light and so frequent, and carried such short distances, that steam locomotives cannot be furnished at lower first cost or operated more cheaply, as in the plain case of a street railway.

(2) Where the gross receipts will be so increased by the change of system and the mode of operation as to more than pay for the increased first cost and operating expenses per car-mile, as in the case of suburban steam systems running out of our great cities and interurban systems between large towns situated a few miles apart.

(3) When competition of parallel electric roads compels the change to save what traffic there is, irrespective of the question whether it will be more profitable.

(4) Where higher speeds are required than can be attained by steam locomotives.

High Speed Railways.

This discussion would not be complete without reference to another field where electric motors will prove a profitable method of operation, namely, (4) where higher speeds are required than can be attained by steam locomotives. This discussion is already long enough, so that the consideration of this must be postponed, but the reader is referred to "Enormous Possibilities of Rapid Electric Travel," by the writer.*

* *Engineering Magazine*, 1897.

Mr. Davis. In that paper is expressed the belief that large centers of population, reasonably near together, will eventually be connected by railways operating at 150 miles per hour or over, with the local street railways as feeders and distributors.

Many more interesting facts and deductions can be obtained from a careful examination and analysis of the curves and tables. They are left to the intelligence of the reader.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will hereafter be reproduced in the Volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

HENRY FLAD, M. Am. Soc. C. E.*

DIED JUNE 20TH, 1898.

Henry Flad was one of many men of rare ability for whose presence in America we are indebted to the Republican Revolution of 1848–49 in Germany. He was born July 30th, 1824, in the Grand Duchy of Baden, near the university town of Heidelberg. His father, Jacob Flad, dying within the same year, his mother, Francisca Brunn Flad, very soon afterward removed to the town of Speyer, a few miles distant, upon the left bank of the Rhine, in the Rhine Palatinate, a province belonging to Bavaria. After passing through the preparatory schools of Speyer, young Henry entered the University of Munich, in Bavaria, where he took the polytechnic course. Upon his graduation in 1846, at twenty-two years of age, he was given a position in the engineering service of the Bavarian Government, his first employment being on works for the improvement of the River Rhine.

The years which immediately followed, particularly 1848 and 1849, were years of great political commotion throughout Europe. Encouraged by the success of the revolution in France, which drove out King Louis Philippe, the longings of the German people for a freer and more united government found such vigorous expression that the princes of the many petty States into which Germany was divided acceded to the convocation of a National Assembly or Parliament, which, in May, 1848, met in Frankfurt to frame a constitution for United Germany. Unfortunately, the deliberations of this assembly showed such wide differences of opinion and so little ability to unite in any workable plan that the ardor of the more conservative classes began to cool. The princes seized their opportunity to reassert themselves, and repudiated the authority of the Parliament.

In Southern Germany, the champions of the Parliament took up arms in its behalf. Among them was Henry Flad, who, then in his twenty-fifth year, joined the Parliamentary army as a captain of engineers. Fortune, however, was against them, and, after several engagements, the Parliamentary army was driven into Switzerland, and disbanded. Meanwhile, its leaders were placed under the ban, and Captain Flad, with many others, was sentenced to death.

* Memoir prepared by Robert Moore, Joseph P. Davis and J. A. Ockerson, Members, Am. Soc. C. E.

Under these circumstances he very naturally turned his face westward, took passage for the United States, and, in the autumn of 1849, landed in New York.

His first employment, after his landing, was as a draftsman in an architect's office. It was not long, however, before he entered the engineering service of the New York and Erie Railroad, then under construction, his headquarters being at Dunkirk, at the extreme western end of the road. Mr. James P. Kirkwood and Mr. James H. Morley, with whom Captain Flad was afterward associated, were also employed at this time on the same road. After the completion of the New York and Erie Railroad, in 1851, we first hear of Captain Flad as located for a time at Tonawanda, between Niagara Falls and Buffalo; and, then, in 1852, as an assistant engineer in the construction of the Ohio and Mississippi Railroad from Cincinnati to St. Louis, his headquarters being at Vincennes, Ind.

Upon the opening of the Ohio and Mississippi Railroad to St. Louis, in 1854—that being the first railway to reach St. Louis from the East—Captain Flad went to Missouri as an assistant engineer on the Iron Mountain Railroad, of which his former colleague on the Erie Railroad, Mr. James H. Morley, was the chief engineer. During the construction of this road, Captain Flad was located at Potosi, Mo. After its completion to Pilot Knob, where for a number of years it ended, he became land and tie agent of the railroad company, with headquarters at Arcadia, Mo. During this period, on September 12th, 1856, Captain Flad was married to Miss Reichard, of St. Louis.

Upon the outbreak of the Civil War, in 1861, Captain Flad came to St. Louis; and enlisted, on June 15th, as a private soldier in Company F, of the Third Regiment, United States Reserve Corps. He was soon promoted Corporal, and then Sergeant.

In July, 1861, a regiment, known as "The Engineer Regiment of the West," recruited mainly in the States of Illinois and Missouri, was organized by Colonel J. W. Bissell, and Henry Flad was made Captain of Company B. In August of the same year he was detailed by General Fremont, then in command at St. Louis, for service in the construction of fortifications at Cape Girardeau, Mo., where he remained for several months. Later in the year, when Fremont was succeeded by General Halleck, Captain Flad was ordered to join General Pope in southeast Missouri, and served as a staff officer through the campaign of New Madrid and Point Pleasant, and at the taking of Island Number Ten, after which he rejoined his regiment at New Madrid. He was with his regiment at Fort Pillow and Pittsburg Landing, and during the operations before Corinth. During the summer of 1862 he was engaged in repairing the Mobile and Ohio Railroad, in building forts at Corinth, and in repairing the Mississippi Central Railroad. He was also engaged in Grant's advance on Grenada. In February, 1863, he was

ordered to Young's Point, where he was employed in engineering work, as he was later at Baxter Bayou, Lake Providence and Bayou Macon.

In April, 1863, he had charge of the repairs of the Memphis and Charleston Railroad at Memphis, Grand Junction, Jackson and Columbus. In October of the same year he was employed in repairing the same railroad east of Corinth, under General Sherman, and was with him at Cherokee, Bear Creek and Iuka, in northern Mississippi.

Meanwhile he had been promoted on November 17th, 1862, to the rank of Major, on July 30th, 1863, to that of Lieutenant-Colonel, and on October 16th, 1863, to that of Colonel. On January 1st, 1864, at Nashville, Tenn., he was transferred, as Colonel, to the First Regiment of Engineers, Missouri Volunteers, a new regiment formed by the consolidation of the former engineer regiment and the Twenty-fifth Missouri Infantry. During the summer of 1864 he was engaged in completing the Nashville and Northwestern Railroad from Nashville to Johnsonville, and in constructing defensive works. In August he was ordered to Atlanta, and served there and in that neighborhood until about the first of November, his last work being the construction of a new line of fortifications at Atlanta.

At that time the term of enlistment of seven companies expired. The command of the remaining five companies then, under the army regulations, devolved upon the Lieutenant-Colonel, and the Colonel was mustered out November 12th, 1864, at Nashville, Tenn. His term of service had been three years and six months, during which time, with not more than a week's leave of absence, he had been constantly in the field.

Upon being mustered out, Colonel Flad returned to St. Louis, and began to look around for employment in his profession. In a short time the agitation for an improved water supply for St. Louis resulted in a State law authorizing the appointment of a Board of Water Commissioners, charged with the duty of making surveys and plans and constructing a new system of water-works for the city. Soon after the organization of the new board, in the spring of 1865, Mr. James P. Kirkwood, who had formerly been chief engineer of the Pacific (now the Missouri Pacific) Railroad, and had just completed the building of new water-works for Brooklyn, N. Y., was appointed chief engineer, and Henry Flad, chief assistant engineer.

Surveys and investigations were at once begun, and, by the end of the year, a plan was presented for new works with intake, settling basins and filter beds at the Chain of Rocks, and a distributing reservoir on what was then known as Rinkel's Hill, on Easton Avenue, near the present city limits. This plan received the approval of the Board of Water Commissioners; and, as subsequent experience has abundantly proven, was undoubtedly the best. But, besides running counter to

some private interests, it involved such a large outlay and such a radical departure from the old plan that, on the part of many leading citizens, as well as the city authorities, it encountered overwhelming disapproval. The opposition finally became so great that the Water Commissioners were called upon by the City Council to resign. To this demand they presently acceded, and, in July, 1866, a new board, committed to a new plan, was appointed. Meanwhile Mr. Kirkwood had been commissioned to go to Europe to study the subject of filtration, and Colonel Flad was left as acting chief engineer. In December, 1866, a revised plan, with intake and settling basins at Bissel's Point and a distributing reservoir on Compton Hill, substantially as afterward built, was presented.

Early in the following year, the act organizing the Board of Water Commissioners was amended, the number of members being reduced from four to three, and in March, 1867, a new board was appointed with Colonel Flad as one of its members. This position he held continuously by reappointment for eight years, or until April, 1875. During that time the new water-works were completed, and during the year 1872 were put into service.

While he was still acting as assistant engineer to Mr. Kirkwood, Colonel Flad made the acquaintance of Captain James B. Eads, who was at that time engaged upon plans for gun carriages and turrets. The rooms occupied by the Water Board being larger than they then needed, Captain Eads, upon his request, had been granted space in which to place a draftsman at work. This was followed by frequent discussions between the two men upon engineering questions, which led to a mutual recognition of each other's abilities and laid the foundation of a life-long friendship. When, therefore, in 1868, Captain Eads, as chief promoter as well as chief engineer, was ready to begin the work of constructing the great bridge over the Mississippi River at St. Louis, he very naturally tendered the position of chief assistant engineer to Colonel Flad. As the duties of the latter as member of the Board of Water Commissioners did not require all his time, this opportunity to take part in this most interesting and important work was gladly accepted, and he retained his connection with it until its completion in 1874. Some of the boldest features of this great enterprise, such as the method of erection without false work, were due to Colonel Flad.

During 1875 and 1876 he was engaged as consulting engineer in various works in conjunction with Mr. Charles Pfeiffer, who had been associated with him on the St. Louis bridge, Mr. Thomas J. Whitman, Chief Engineer of the Water-Works, and Professor Charles A. Smith, of Washington University. Among his other engagements was that of engineer for the commissioners who purchased and laid out Forest Park, St. Louis.

In the autumn of 1876, the new charter of the City of St. Louis was inaugurated, and Colonel Flad was elected the first President of the newly constituted Board of Public Improvements. This office he held continuously for nearly fourteen years, being re-elected in 1881, 1885 and 1889.

The problem to which the new Board addressed itself was that of taking the whole system of municipal public works out of the mire of politics, and placing it upon the basis of merit and fitness. Into this work Colonel Flad entered with characteristic zeal and a determination which nothing could shake. His efforts were crowned with entire success, so that during the whole period of his administration the Board over which he presided had the entire confidence of the whole community. Every citizen felt sure that in every department of the public works the city received a dollar's worth for every dollar spent, and in this respect St. Louis became a model for other cities.

In April, 1890, he resigned his office as President of the Board of Public Improvements to accept an appointment as member of the Mississippi River Commission, a position in which he remained until his death.

Into this new field of work he entered with characteristic zeal and energy. In November, 1891, he was appointed a member of a committee of the Commission to prepare a project for the construction of a dredge boat to be used in deepening the Mississippi River over the bars in extreme low water—a problem in which his remarkable mechanical ingenuity found a fertile field. After a most careful study of the subject in all its bearings, plans were drawn for an experimental dredge, which was built under his personal supervision.

While this experimental dredge was by no means perfect, its main features were found to meet the requirements so well that they have been embodied in all the later dredges built by the Commission, and its construction marks the beginning of a new era in the history of river improvement.

Colonel Flad became a member of the American Society of Civil Engineers on February 15th, 1871, and was President of the Society for the year ending January 19th, 1887. He was also a charter member of the Engineers' Club of St. Louis, and for twelve years, from 1868 to 1880, was its President, thus receiving from both organizations the highest honor within their power to bestow.

His death occurred June 20th, 1898, at Pittsburg, Pa., where, on his way home from a meeting of the Mississippi River Commission, he stopped to visit Mr. Godfrey Stengel, a life-long friend who, forty-nine years before, had come with him on the same ship to America. He died very suddenly of acute heart failure while walking home from one of the parks in company with Mr. and Mrs. Stengel. Up to the last moment he was in excellent spirits and died without pain, as without fear.

As an engineer, Colonel Flad was remarkable for his great fertility of invention. For every new problem he had not only one, but many solutions, and the rapidity with which he grasped all its conditions and framed his plans to meet them amounted to genius. In doing this he was not limited by precedent, but looked instinctively for new and better methods than any before known. In boldness and originality he has had but few equals in the annals of the profession. And, like the most successful workers in every field, he delighted in his work for its own sake. Nothing could exceed the interest with which he attacked a new problem, and he gave himself no rest until he had solved it. The solution once found, however, the whole subject ceased to interest him, and he passed on to something new. This trait is illustrated by the fact that, although he took out numerous patents for new and useful inventions, to their introduction and utilization he gave no thought. It was the work, rather than its rewards, for which he cared.

As a man he was equally great. His unassuming modesty, his perfect candor and simplicity, his unflinching courage, his absolute fidelity to his convictions, his single-minded subordination of personal to the public welfare—qualities which were written in every line of his face and manifested in every act of his life—all stamped him as a man of the highest type. Anyone who knew him believed in him—without limit. His name was a synonym for fidelity and skill, and all knew that every work committed to his charge would be well done and come from his hand as sound and flawless as the man himself.

This evident and perfect integrity of purpose made his public service a legacy of incalculable value to his fellow citizens. His life was a demonstration of how honorable the public service can be made, and is an encouragement to those who have not yet lost faith in the possibility of having this service in all its branches lifted to the same standard to "abate no jot of heart or hope," but still work on for the accomplishment of this high end.

To his fellows in his chosen calling, his name and example are especially precious; for in him was realized the highest ideal of the engineer—a man of trained intellect, controlled by an iron will and directed to the noblest public ends. And his success in attaining this ideal will inspire others to frame their lives upon the same noble lines.

HISTORICAL SKETCH

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS,

By Charles Warren Hunt, M. Am. Soc. C. E.

Cloth, 6x9 Inches.

Printed by order of the Board of Direction of the American Society of Civil Engineers, to be sold only on subscription. The proceeds to be devoted exclusively to the fund for the New Society House.

At the Annual Meeting, January 19th, 1898, the following facts in regard to the subscription to this book were brought out:

Two thousand copies were printed; 300 were bound in full morocco, of which 216 have been sold at $10 per copy, the resulting net profit being $943.06. Seventeen hundred copies, which have been paid for, are still on hand, and the Board of Direction was requested to consider the propriety of offering to the membership these copies bound in a less expensive style and at a reduced price, the net proceeds to be applied to the building fund.

In compliance with this request it has been decided to bind as many copies as are necessary to supply the demand, in a handsome cloth binding and to supply them at $5 per copy.

This action has been taken in the belief that many members will welcome the opportunity of contributing something to the building fund.

There are a few copies still on hand of the first lot bound and these can be obtained by those who so desire at $10 per copy.

Orders should be sent to the Secretary.

The book begins with a brief statement of the first movement to form a National Society of American Engineers in 1839. The organization of the American Society of Civil Engineers and Architects in 1852 is then described, a list of its promoters and charter members given, and the work accomplished in its first two years of life sketched. The reorganization of the Association in 1867 and the important events in its career from that date to 1873, when the first publication was issued, are then given in chronological order. Succeeding chapters are under the following heads: Locations Occupied by the Society; Library; International Exhibitions; Publications; Badge; Constitutional Changes and Work Accomplished. Under the head of "Comparative Growth of National Engineering Societies" short sketches of the Institution of Civil Engineers and the Société des Ingénieurs Civils are given. The illustrations consist of 35 half-tone portraits of past officers of the Society and one diagram, all handsomely printed on heavy paper.

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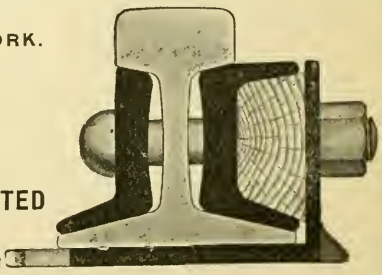

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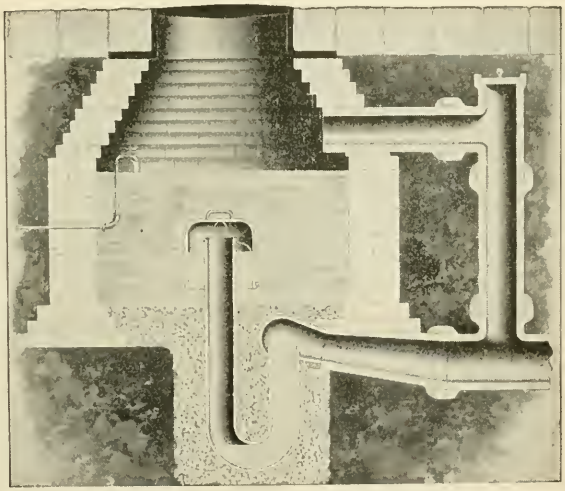

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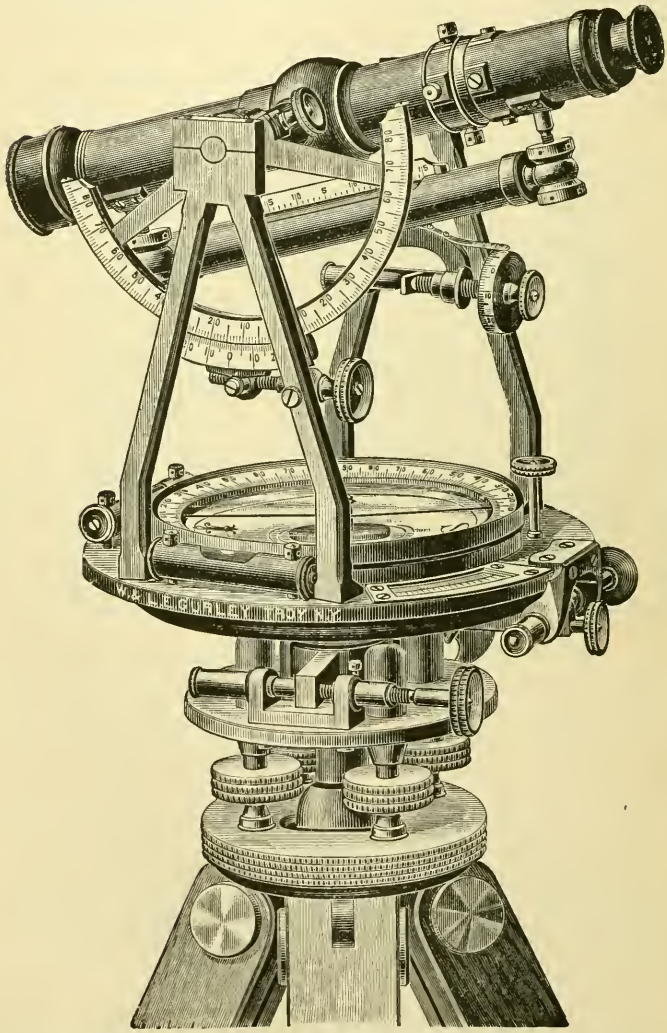

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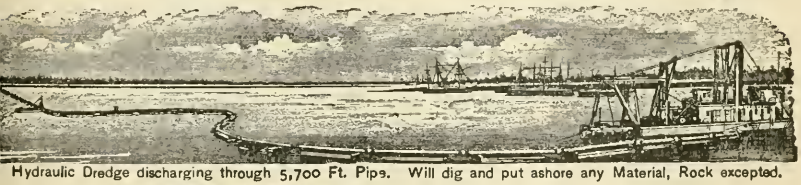

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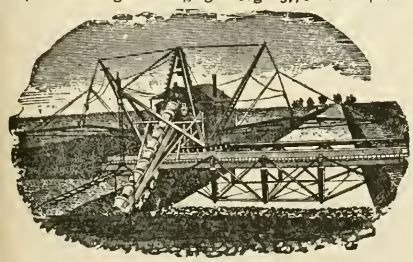

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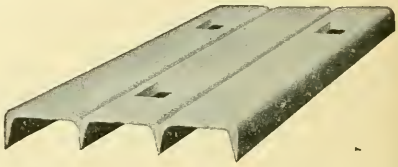

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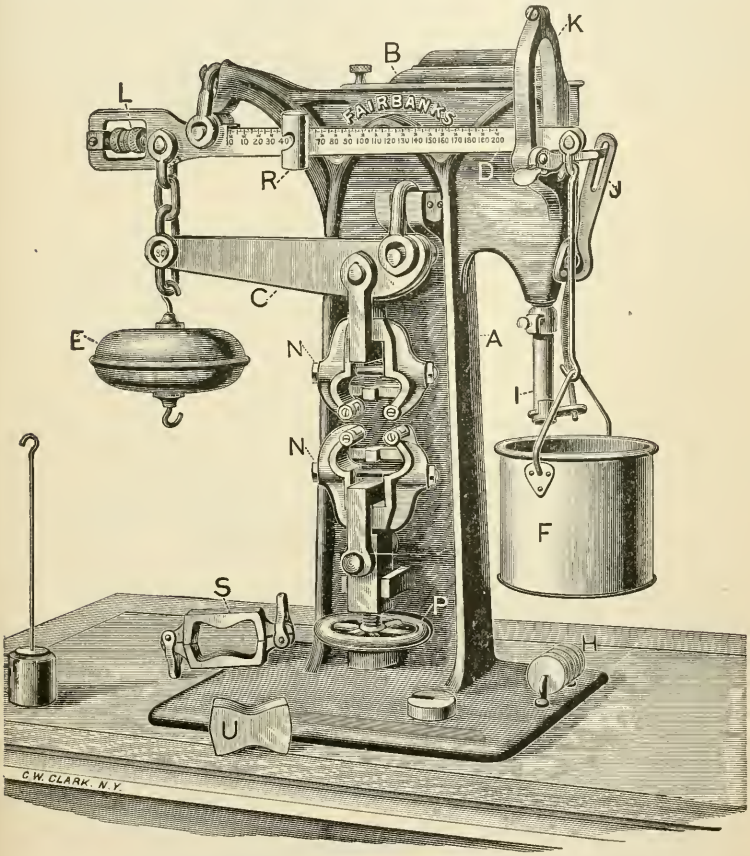

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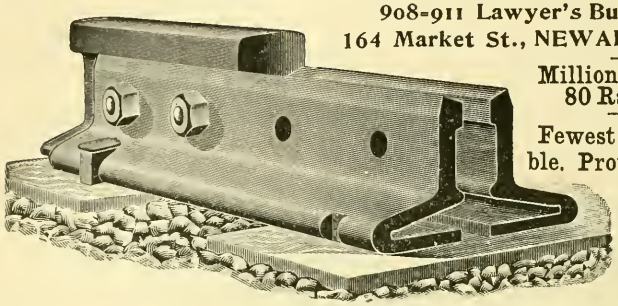

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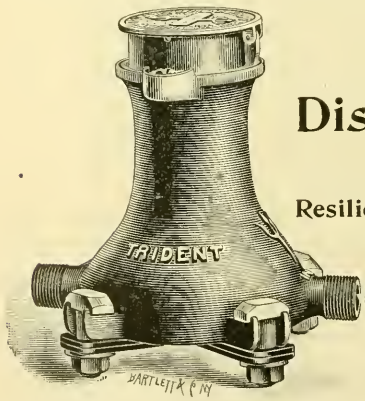

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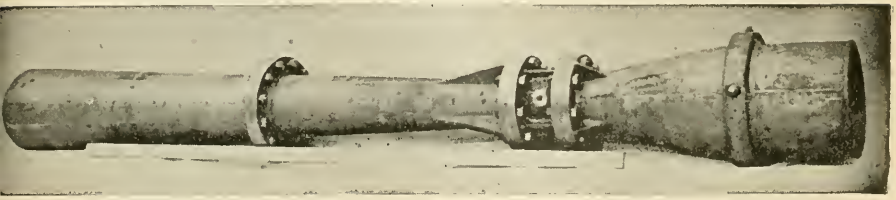

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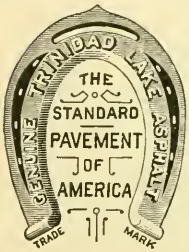

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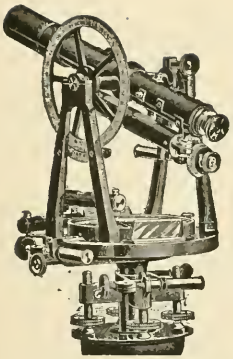

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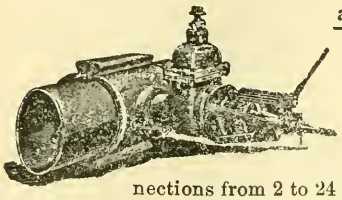

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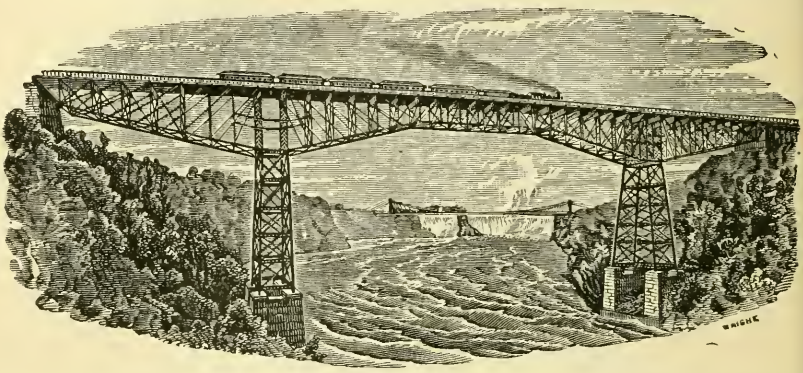

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